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UNDERSEEPAGE CONTROL AT FORT RANDALL DAM

S. T. Thorfinnson,[1] A. M. ASCE

SYNOPSIS

This paper describes a relief well system installed to control underseepage beneath Fort Randall Dam which is located on the Missouri River in southeastern South Dakota. Data are presented on the field investigations made, the design of the wells and the performance of the completed system.

INTRODUCTION

Fort Randall Dam is located on the Missouri River in southeastern South Dakota, as shown in Fig. 1, and is an integral part of the Pick-Sloan Plan for Missouri River development. It is a multi-purpose project furnishing flood control and navigation storage, generating 320,000 Kw of power and providing recreational facilities. Construction of the dam was started in 1946 and completed in 1955.

Control of underseepage in the valley alluvium beneath the dam was provided by an impervious upstream blanket and a system of relief wells at the downstream toe of the main impervious embankment. Several other installations were made to control underseepage during and subsequent to construction in relation to the various structures but are not considered within the scope of this paper. These installations included vertical drains to control uplift in the Niobrara chalk beneath the spillway crest structure; relief wells to control uplift pressures in the Codell sand-stone beneath the spillway and outlet works stilling basins; and drains to control seepage through the Niobrara chalk surrounding the power and flood control tunnels.

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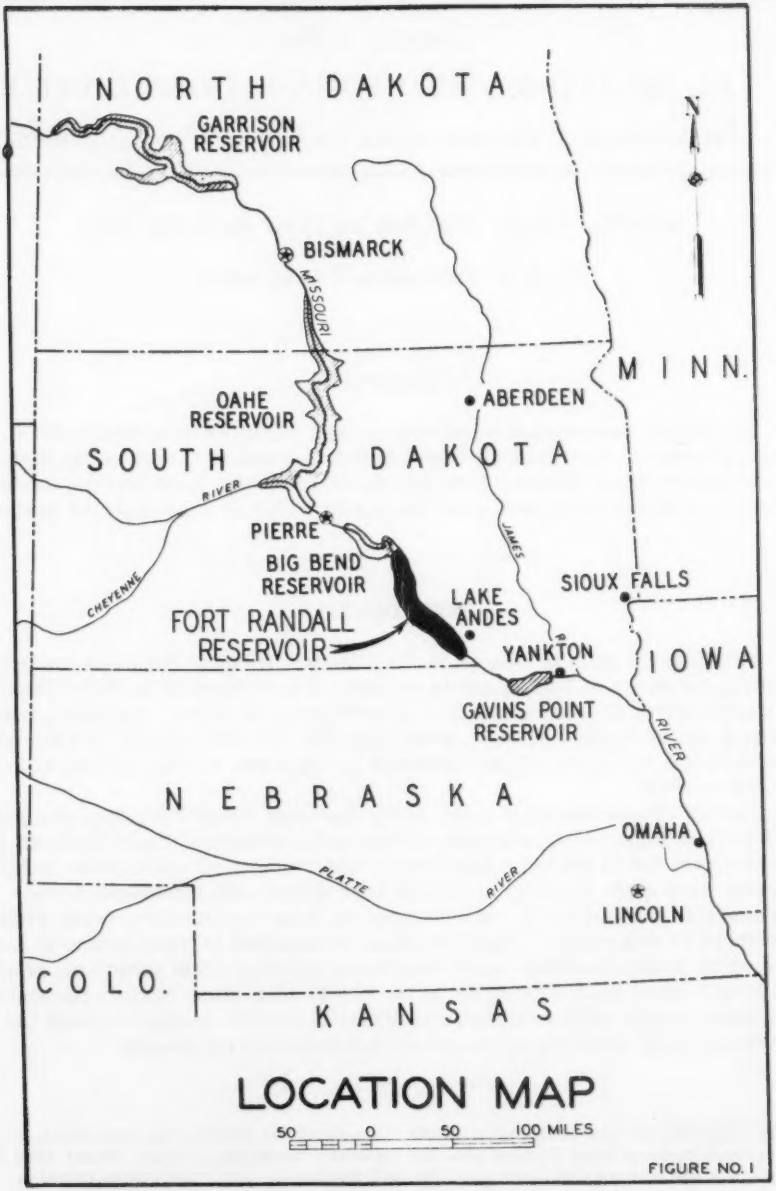

NORTH DAKOTA
GARRISON RESERVOIR
BISMARCK
MISSOURI
N
MINN.
OAHE RESERVOIR
ABERDEEN
SOUTH DAKOTA
RIVER
PIERRE
JAMES
BIG BEND RESERVOIR
SIOUX FALLS
CHEYENNE
LAKE ANDES
FORT RANDALL RESERVOIR
YANKTON
IOWA
GAVINS POINT RESERVOIR
RIVER
NEBRASKA
OMAHA
PLATTE
RIVER
LINCOLN
COLO.
KANSAS
LOCATION MAP
50 0 50 100 MILES

FIGURE NO. 1

General

A plan showing the dam and pertinent structures is shown on Fig. 2. The power and flood control structures as well as the spillway are located on the left abutment. The dam which is of rolled earth construction, extends across the valley approximately 10,000 feet to the right abutment. The maximum height of the dam is 160 feet, the crest width is 60 feet, and the maximum width at the base, including the upstream impervious blanket and the downstream chalk berm is about 5000 feet. Approximately 28,000,000 cubic yards of impervious fill and 22,000,000 cubic yards of chalk berm were utilized in its construction. The majority of the impervious fill was obtained from excavation for the structures in the left abutment glacial tills supplemented by loess material borrowed from the right bank. The chalk was obtained from required excavation on the left bank.

A general picture of the geology at the dam site is shown on Fig. 3, which is a profile on the centerline of the dam. The bedrock encountered at the site consists of the Pierre shale, Niobrara chalk, Carlile shale, and Green horn Limestone in descending order. The outlet works, powerhouse and spillway structures were located on the left abutment to take advantage of the relatively high elevation of the Niobrara chalk as a foundation material and as a tunnelling medium for the power and flood control tunnels.

The river valley is approximately 8,000 feet wide at the site, 4,000 feet of which is occupied by the existing river channel adjacent to the left abutment. The remaining 4000 feet consists of a river terrace. The surface of the terrain on the right abutment rises abruptly approximately 60 feet adjacent to the river and then slopes gently upwards towards the hills south of the river. The overburden material on the left abutment is a glacial drift deposit reaching depths of approximately 100 feet. The river is underlain by alluvial sand deposits reaching a maximum depth of 150 feet. A deposit of loess soil, reaching a maximum depth of about 50 feet, caps the right terrace adjacent to the river. The loess is underlain by a deposit of poorly consolidated clays which reaches maximum depths of about 130 feet in an old buried channel of the river.

The clay deposits are thin adjacent to the river and are underlain by pervious sands and gravels which overlay a relatively high "Chalk island" between the old buried channel and the river.

Typical sections of the embankment are shown on Fig. 4. On the left abutment the embankment consists in part of in-place glacial till and in part of rolled earth fill. An upstream impervious zone with a shallow cutoff trench into the chalk and a downstream toe drain were provided in view of the existence of some relatively pervious layers of sand and gravel in the glacial till. The upstream slopes are protected from wave action by dumped riprap. The valley section of the dam consists of an impervious main embankment flanked by chalk berms with an impervious blanket extending upstream beneath the chalk berm. Valley alluvium exposed in the side slopes and bottom of the outlet works approach channel were also blanketed to maintain the integrity of the "upstream blanket." The upstream chalk berm was constructed with flat beach slopes in lieu of hard rock riprap for wave protection in the range of pool fluctuation. The embankment section on the right abutment consists of an impervious main embankment with an upstream chalk berm. The thick deposit of loess was removed beneath the main embankment to minimize differential settlement. An impervious cutoff was provided through the

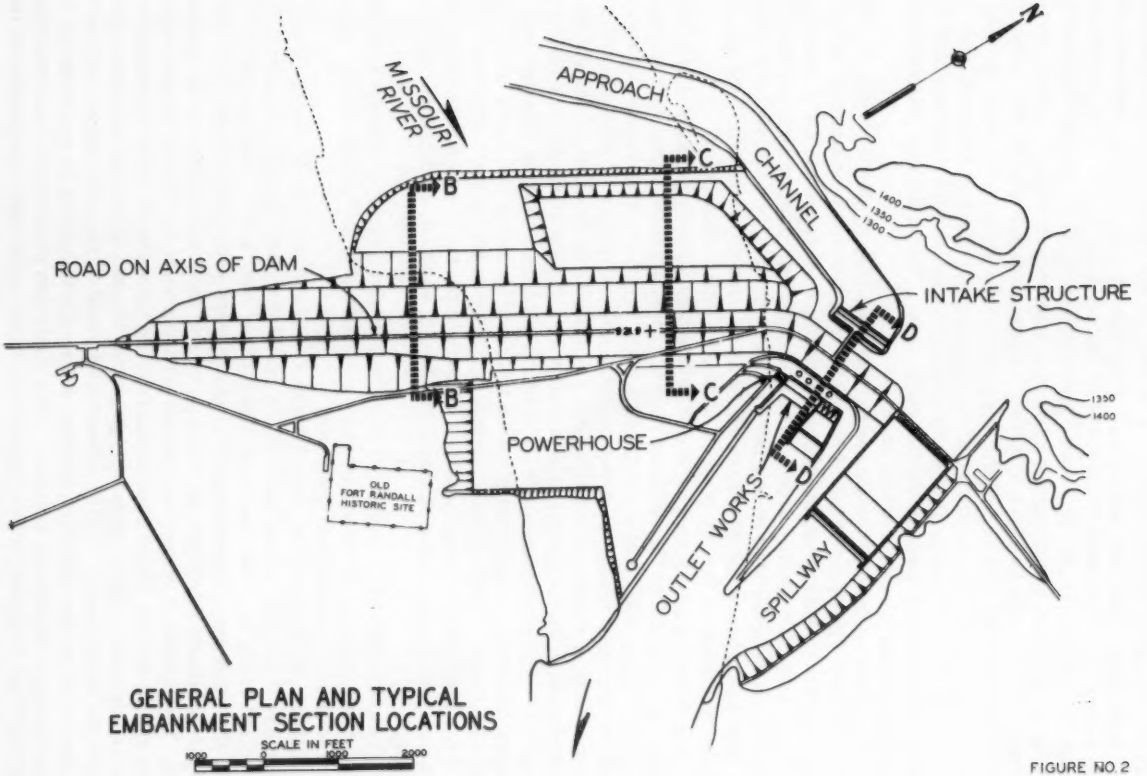
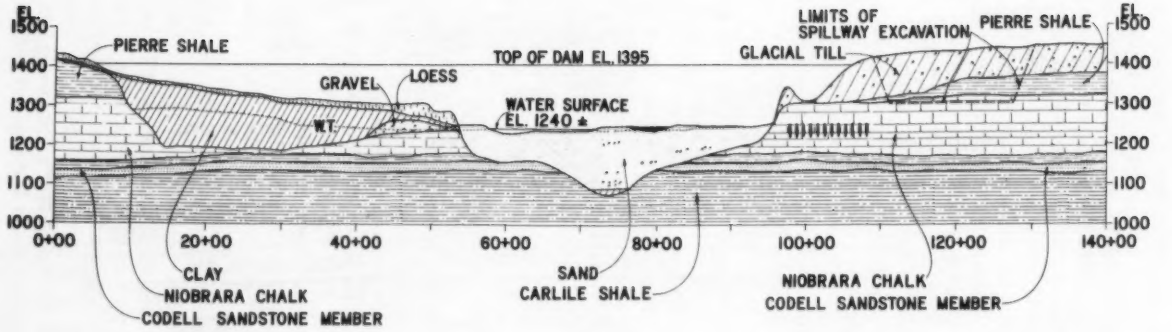
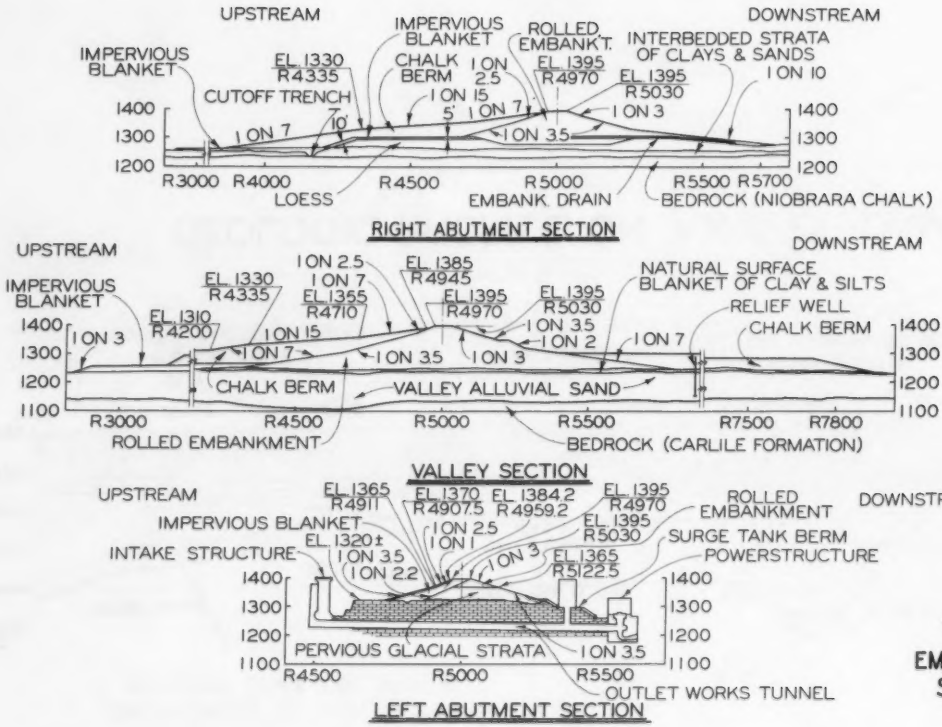

MISSOURI RIVER
APPROACH
CHANNEL
ROAD ON AXIS OF DAM
INTAKE STRUCTURE
POWERHOUSE
OLD FORT RANDALL HISTORIC SITE
OUTLET WORKS
SPILLWAY
A
B
C
D
1400
1350
1300
1330
1400
N
GENERAL PLAN AND TYPICAL EMBANKMENT SECTION LOCATIONS
SCALE IN FEET
1000 0 1000 2000

FIGURE NO. 2



GEOLOGIC PROFILE ON AXIS OF DAM

FIGURE NO. 3



TYPICAL EMBANKMENT SECTIONS

FIGURE NO. 4

pervious sand and gravel layer overlying the "chalk island" and a pervious embankment drain was included under the downstream portion of the embankment. The top few feet of loess upstream of the main embankment was removed and recompacted to reduce the relatively high vertical permeability of the natural material.

Prior to closure and diversion of the river through the tunnels, which was accomplished during the summer of 1952, three lines of well point type piezometers were installed through the embankment into the valley alluvium to measure hydrostatic pressures in the pervious foundation and to facilitate an estimation of conditions without relief wells as well as the determination of the effectiveness of the upstream blanket.

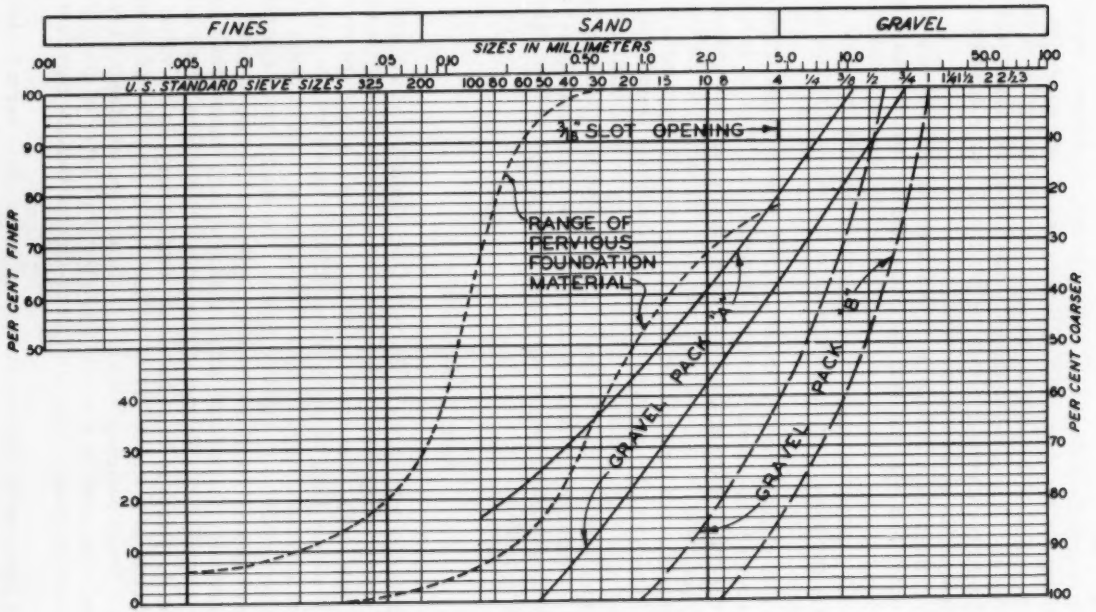
Piezometer readings taken during the late winter and early spring of 1953, when 28 feet of water had been ponded, indicated that the installation of relief wells would be desirable to reduce the possibility of the development of excessive hydrostatic pressures in the valley alluvium beneath the chalk berm, and thus minimize the possibility of movement of the valley alluvium soils up into the chalk berm.

Three relief well plans were considered, i.e.: (1) relief wells at the toe of the main impervious embankment with the discharge carried by an underground conduit, (2) relief wells at the toe of the main impervious embankment discharging into the chalk berm, and (3) relief wells at the toe of the chalk berm. Plan 2 was selected because location of the wells at the downstream toe of the main impervious embankment was most desirable from the standpoint of achieving maximum pressure relief. Pumping in tests indicated that the chalk berm was very pervious in the horizontal direction thus eliminating the necessity of an underground conduit which was not considered practical due to the difficulties of accurately computing the discharge to be carried and maintenance of the conduit.

Field Investigations

The foundation soils in the area of the relief well installation are generally water deposited sands and gravels with some clay at the surface between stations 80 + 00 to 87 + 00. The sand varies in thickness across the river bottom from a relatively shallow depth adjacent to the abutments to a maximum thickness of approximately 150 feet. The upper portion of valley alluvium consists primarily of fine to medium sands. The sands become somewhat coarser and in part gravelly with increasing depth, but the fine to medium sands still predominate. Occasional small seams of silty sands, chalk and lignite float are encountered. Ranges in gradation of the valley sands are shown on Fig. No. 5. Records from previous pumping tests in the valley sands indicated a coefficient of permeability in the horizontal direction of about 0.002 ft. per sec. These pumping tests also indicated the existence of two distinct acquifers, separated by a relatively impervious layer of clay. However, numerous borings made in the alluvium indicated that the clay layer was not continuous.

A drilling and pumping-in test was conducted in the chalk berm to determine (1) whether the dumped chalk material could be drilled with a large diameter bit, (2) whether the walls of the hole would stand without casing, (3) the quantity of gravel that could be flushed into the voids in the chalk berm adjacent to the hole, (4) the resistance of flow of water out of the well screen,



GRADATION OF VALLEY SANDS AND
RELIEF WELL FILTER MATERIALS

FIGURE NO. 5

and (5) the amount of settlement and caving which might occur in the vicinity of the well due to movement of a large quantity of water.

The hole was drilled with a 39-inch auger bit. Although the chalk berm had been placed by dumping in 10 foot lifts, the hole remained open to its full depth of approximately 50 feet. The average rate of drilling was 0.1 foot in 7 minutes of actual drilling time. The hole was cased with 35-inch diameter casing. A 27.5 foot length of 6+ inch screen consisting of slotted metal pipe was centered and set to the bottom of the hole. The 1/4 inch wide slots were cut parallel to the axis of the pipe, were 2 feet long and overlapped 4 inches. Seven slots were cut around the circumference of the pipe. An 8-inch riser pipe extending to the surface of the berm was split and welded to the slotted pipe to form a water tight joint.

The annular space between the screen and the chalk was gravel packed by flushing the gravel down a 3-inch pipe, withdrawing the 35-inch casing as the gravel level rose. A total of 36 cubic yards of gravel was used approximately 21 of which was flushed into voids in the chalk berm adjacent to the hole. Water was pumped down the hole at an average rate of 232 g.p.m. for 107 hours during the gravel packing operation. Subsequent to completion of the gravel packing, water was pumped into the well screen at the rate of 430 g.p.m. for a period of approximately 19 hours. No appreciable head was built up in the screen during the test.

Relief Well Design

It was assumed, for the purpose of computing well spacing, that hydrostatic uplift would be resisted only by the weight of a saturated impervious blanket five feet in thickness and of uniform extent across the valley. This assumption was not fully borne out by the logs of borings, but was considered justified in view of the uniformity across the valley indicated by the piezometer observations. In areas where the borings did not indicate the existence of the impervious blanket, stratification in the upper portion of the valley sands apparently acted as blanket in resisting upward flow. The weight of the chalk berm was not considered as resisting upward flow. The weight of the chalk berm was not considered as resisting the uplift pressures as it was assumed that piping could occur in the large voids in the dumped chalk. Based on a unit weight of saturated blanket material of 115 pounds per cubic foot and a factor of safety of 1.0, the maximum allowable uplift pressure at the base of the blanket was computed to be 9.2 feet of water.

Three lines of well point type piezometer tubes had been installed in the pervious valley alluvium normal to the axis of the dam as shown on Fig. No. 6, to measure hydrostatic foundation pressures. The piezometers penetrated the main embankment and the upstream and downstream chalk berms. Embankment sections showing the three lines of piezometers are shown on Figs. No. 7, 8 and 9. These figures also show typical piezometric observations made in the design period. The effective resistance of the impervious upstream blanket was determined by extending the piezometric gradient upstream until it intersected the water surface in the pool. The ratio of piezometric head to the total head in the pool, referenced to a tailwater elevation datum showed considerable consistency in lines parallel to the axis of the dam, indicating that the effective resistance of the upstream blanket was uniform across the valley. The total effective resistance of the upstream blanket

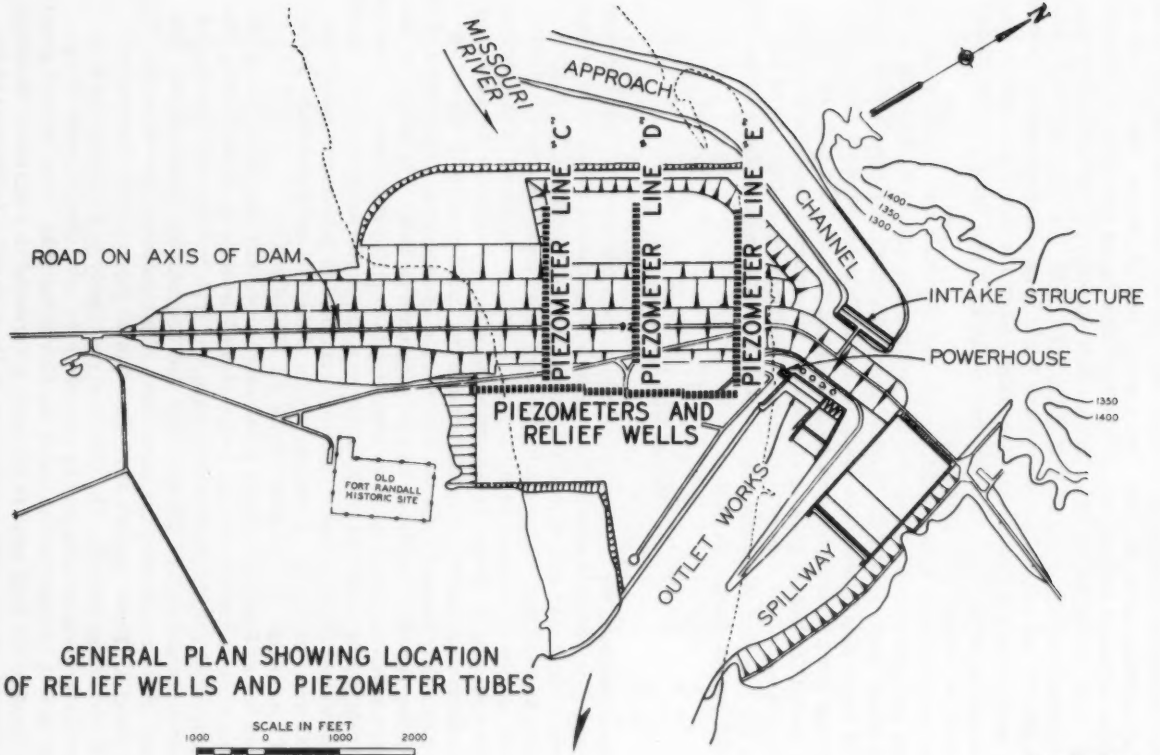
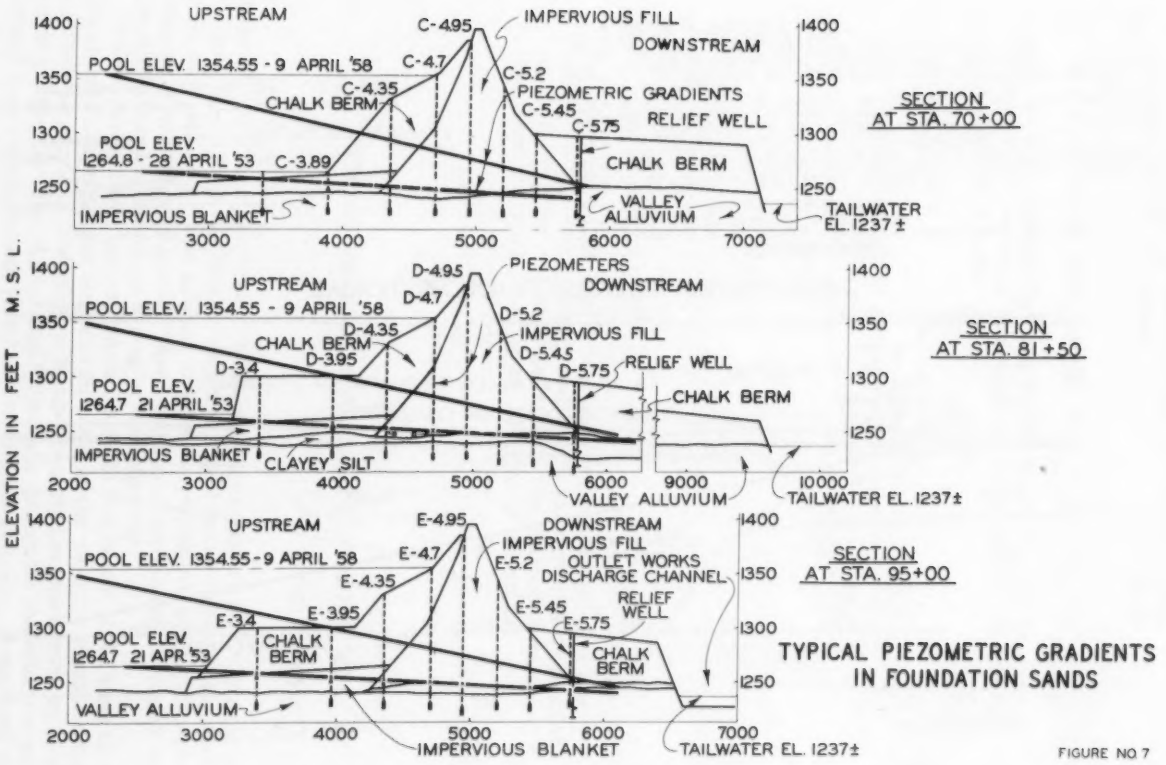
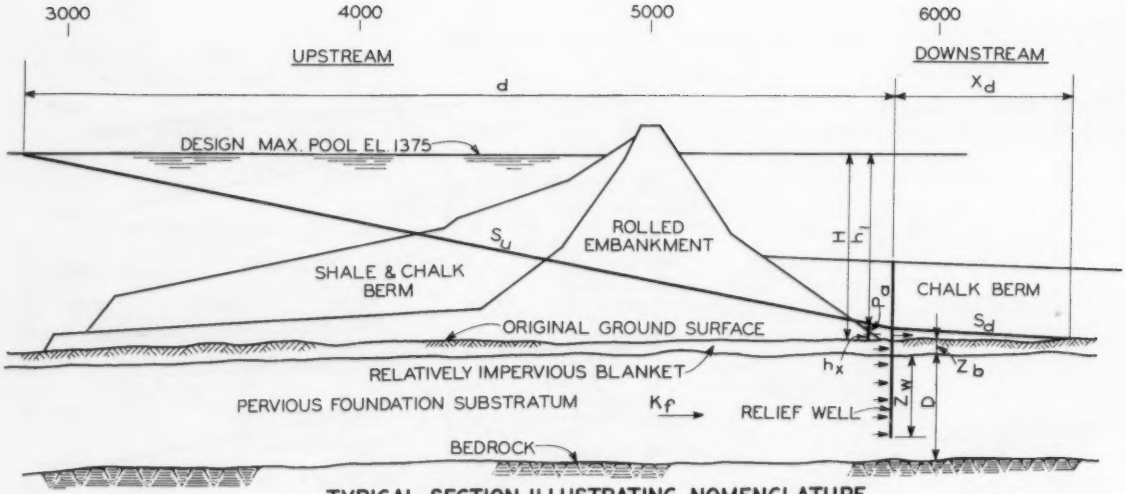

MISSOURI RIVER
APPROACH
"C"
"D"
"E"
PIEZOMETER LINE
PIEZOMETER LINE
PIEZOMETER LINE
CHANNEL
ROAD ON AXIS OF DAM
INTAKE STRUCTURE
POWERHOUSE
PIEZOMETERS AND RELIEF WELLS
OLD FORT RANDALL HISTORIC SITE
OUTLET WORKS
SPILLWAY
1400
1350
1300
1350
1400
N
GENERAL PLAN SHOWING LOCATION
OF RELIEF WELLS AND PIEZOMETER TUBES
SCALE IN FEET
1000 0 1000 2000

FIGURE NO. 6



TYPICAL PIEZOMETRIC GRADIENTS IN FOUNDATION SANDS

FIGURE NO 7



TYPICAL SECTION ILLUSTRATING NOMENCLATURE

WELL SPACING FORMULAE

1. FULL PENETRATING WELL FORMULA = $P_a = \left(\frac{a}{2\pi} \log_e \frac{a}{2\pi r_w}\right) S = F.S.$

2. MEAN POTENTIAL FORMULA FOR PARTIALLY PENETRATING WELLS $P_a = \left[\frac{aD'}{2\pi Z_w'} \log_e \frac{a}{2\cdot\left(\frac{D'}{Z_w'}-1\right)\left(1-\frac{D'}{2Z_w'}\right)\pi r_w}\right] S$

3. MID-POINT POTENTIAL FORMULA FOR PARTIALLY PENETRATING WELLS. $P_m = \left[\frac{aD'}{2\pi Z_w'} \log_e \frac{a}{2\left(1+\frac{D'}{Z_w'}\right)\pi r_w}\right] S$

NOMENCLATURE

Z_b = THICKNESS OF RELATIVELY IMPERVIOUS DOWNSTREAM BLANKET
D = THICKNESS OF PERVIOUS SUBSTRATUM
Z_w = DEPTH OF PROPOSED RELIEF WELL
H = TOTAL HEAD
P_a = MEAN POTENTIAL OVER PLANE OF WELLS
P_m = SURFACE POTENTIAL AT MID-POINT BETWEEN PARTIALLY PENETRATING WELLS
H_x = HEAD LOSS DUE TO FLOW OUT OF WELL SCREEN INTO CHALK BERM
H_1 = $H-(P_a+h_x)$ = TOTAL HEAD LOSS FROM SOURCE TO LINE OF WELLS
d = EFFECTIVE RESISTANCE UPSTREAM OF LINE OF WELLS
X_d = EFFECTIVE RESISTANCE DOWNSTREAM OF LINE OF WELLS
S = $S_u - S_d$ = NET POTENTIAL GRADIENT PRODUCING DISCHARGE FROM WELLS
a = WELL SPACING
h_x = 0.11 $a S$ = POTENTIAL MIDWAY BETWEEN FULLY PENETRATING WELLS IN EXCESS OF THE AVERAGE POTENTIAL OVER THE PLANE
K_f = HORIZONTAL PERMEABILITY OF PERVIOUS SUBSTRATUM
Q_w = $K_f D a S$ = DISCHARGE OF WELL
r_w = RADIUS OF PROPOSED WELL

FIGURE NO. 8

Design Data

Average original ground surface elevation = 1235.
Average bedrock elevation = 1165.
H = 140 ft.
D = 65 ft.
Full penetrating well formula = $P_a = \left(\frac{a}{2\pi} \log_e \frac{a}{2\pi r_w}\right) S = F.S.$

Assume well spacing "a" and value for $P_a + h_x$ and compute total uplift by trial and error. Total uplift $(P_a + h_x + h_3) \geq$ Allowable uplift.

Assume a = 100

	1st Trial	2nd Trial
$P_a + h_x$ (Assumed) =	3.2	3.7
h_1 =	136.8	136.3
S_r =	.0428	.0426
S_1 =	.0080	.0093
S =	.0348	.0333
Q_w (In c.f.s.) =	.45	.43
h_x =	1.95	1.89
$P_a = F S = 55 S$ =	1.91	1.83
$P_a + h_x$ =	3.86	3.72
h_3 =	.38	.37
$h_x + P_a + h_3$ =	4.24	4.09

The computed value for $P_a + h_x$ corresponds to the assumed values in the second trial and the total computed potential $h_x + P_a + h_3$ is slightly less than the allowable value, therefore the well spacing of 100 ft. is adequate.

TYPICAL COMPUTATION FOR FULLY PENETRATING WELL

FIGURE NO. 9

measured from the line of relief wells, was approximately 3200 feet as shown on Figs. No. 7, 8 and 9. This value which would be the same for any pool level, was used in the well spacing and discharge computations. This resistance is considerably greater than computed from blanket formulas and was attributed to siltation occurring in the reservoir.

The reservoir level had not reached a sufficient height at the time of design of the wells to permit the determination of the effective resistance of the relatively impervious downstream blanket beneath the chalk berm by piezometric gradients. Use of a blanket formula to determine the effective resistance was not considered justified due to non-uniformity of the natural blanket. As a result the effective radius of a pumped well was used in the well spacing computations for the resistance of the downstream blanket. An

average value of 400 feet was adopted based on several pumping tests that had been made in the valley alluvium.

The wells used in this installation were unlike a standard relief well where the head is immediately dissipated at the surface. In this system the water is discharged through the well screen into the chalk berm and a head is developed in the screen of sufficient magnitude to overcome the head losses through the slots. The amount of head loss was computed considering the slots as free flowing submerged orifices. The degree of clogging of the slot openings could only be approximated and the conservative assumption was made that only 25 per cent of the slot openings were effective. Considering a well screen with 30 square inches of slot opening per lineal foot, an orifice discharge coefficient of 0.61 and assuming the slop openings 25 per cent effective, the head loss was computed using the following formula:

$$h_x = (5.89 Q_w)^{2/3}$$

wherein Q_w is the total well discharge. For the purpose of design computations the screen entrance and riser loss and the head built up in the chalk berm were considered negligible.

Relief wells which fully penetrated the valley alluvium were installed at all locations except in the center of the valley section where the bedrock is at a considerable depth. The well spacing for fully penetrating wells was computed by the method developed by Mr. P. T. Bennett,(1) M. ASCE. The well spacing was computed so that the maximum head midway between the wells would not exceed the aforementioned allowable uplift pressure. Although the well spacing was computed for fully penetrating wells, they were stopped 10 feet above bedrock due to the existence of a boulder layer immediately overlying the bedrock.

Partially penetrating relief wells, penetrating to El. 1150 or about 60 per cent of the depth of the valley alluvium were used in the center of the valley section where bedrock is at considerable depth. For the purpose of well spacing computations, the valley alluvium was assumed to be 16 times more permeable in the horizontal direction than vertically. The depth of well and the permeability were transformed by the relationship.

$$\sqrt{K_f / K_v}$$

The well spacing was computed using the partial penetration multiple well spacing formula developed by P. T. Bennett.(2) These formulas and those for fully penetrating wells are shown on Fig. No. 10.

A typical relief well is shown on Fig. No. 11. Discharge computations indicated that 8-inch I.D. screen and riser pipe would provide adequate capacity to carry the flow without excessive head loss. Treated wooden well screen and riser pipe were chosen for their advantage of low cost and long life as long as the wood is immersed. The well screen pipe was slotted with 3/16 inch slots of such spacing and length to provide a minimum of 30 square inches of clear slot opening per lineal foot of screen. The screen section is continuous through the valley alluvium and well up into the chalk berm.

The range of gradation of the filter material for that portion of the well in the valley alluvium is designated as gravel pack "A" and is shown on Fig. No. 5, together with gradation curves for the limiting sizes of the pervious

Design Data

Average ground surface elevation = 1238.
Average bedrock elevation = 1080.
Bottom of well elevation = 1150.
H = 137 ft.
D = 153 ft. - Transformed D' = 612 ft.
Z_w = 83 ft. - Transformed Z_w' = 332 ft.

Mean potential formula for partially penetrating wells.

$$P_a = \left[\frac{aD'}{2\pi Z_w'} \log_e \frac{a}{2'-(\frac{D'}{a}-1)(1-\frac{Z_w'}{D'})\pi f_w}\right] S$$

Mid-point potential formula for partially penetrating wells.

$$P_M = \left[\frac{aD'}{2\pi Z_w'} \log_e \frac{a}{2\frac{(1-Z_w')}{D'}\pi r_w}\right] S$$

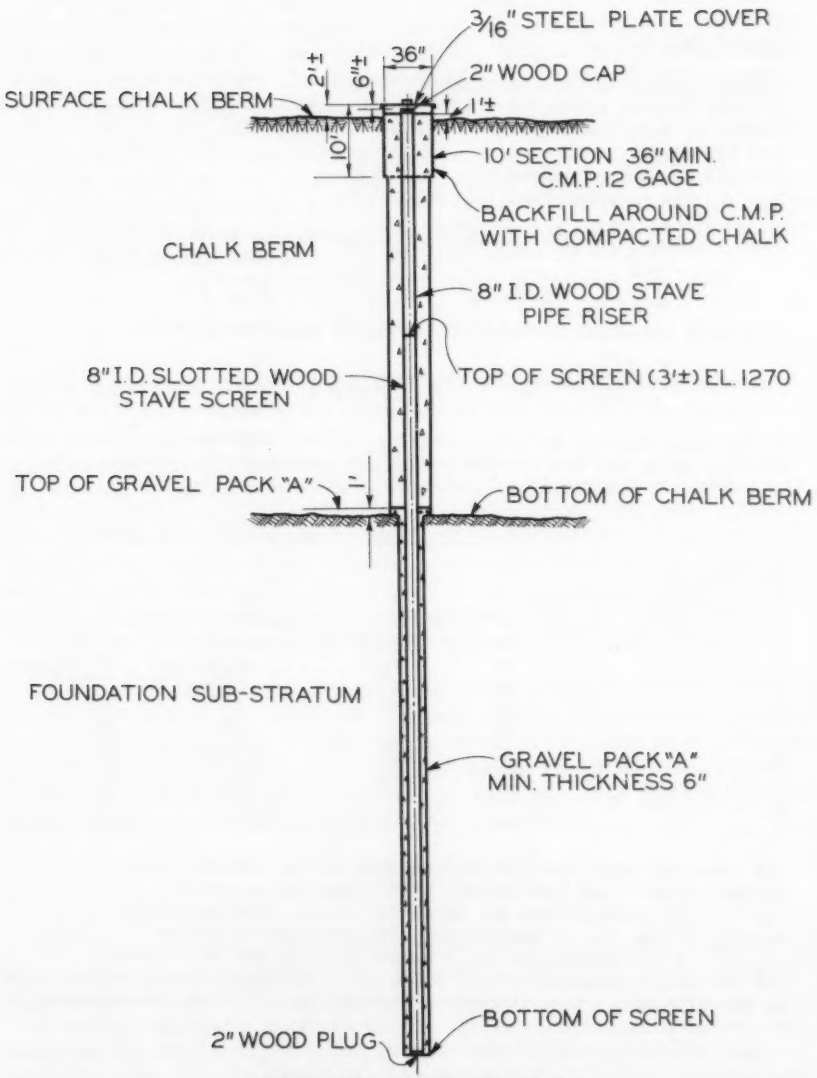
Assume well spacing "a" and value for $P_a + h_x$ and compute total uplift, using the transformed depths and permeability by trial and error. Total uplift $P_M + h_x \leq$ allowable uplift.

	1st. Trial	2nd Trial	3rd Trial
a (Assumed) =	100	100	70
$P_a + h_x$ (Assumed) =	6.0	6.4	5.4
h_i =	131.0	130.6	131.6
S_r =	.0409	.0408	.0411
S_i =	.0150	.0160	.0135
S =	.0259	.0248	.0276
Q_w =	.79	.76	.59
h_x =	2.79	2.71	2.30
P_a =	3.83	3.70	3.14
$P_a + h_x$ =	6.62	6.41	5.44
P_M =		2.80	1.95
$P_M + h_x$ =		5.51	4.25

The computed value for $P_a + h_x$ corresponds to the assumed value in the second trial however the total computed potential, $P_M + h_x$ is greater than the allowable value, therefore, the spacing of 100 ft. is inadequate. The computed value for $P_a + h_x$, corresponds to the assumed value in the third trial and the total computed potential $P_M + h_x$ is approximately equal to the allowable value, therefore the spacing of 70 ft. is adequate.

TYPICAL COMPUTATION FOR PARTIALLY PENETRATING WELL

FIGURE NO. 10



TYPICAL RELIEF WELL

NO SCALE

FIGURE NO. 11

valley alluvium. The gradation of gravel pack "A" was based on standard Corps of Engineers procedure set forth on the Figure. A minimum thickness of gravel pack of 6 inches was provided between the well screen and the foundation material. The wells were surged by intermittent pumping to develop and stabilize the gravel pack. Normally the surging would have been accomplished using a plunger, but a plunger was not considered feasible in this system as the water table was within the screen portion of the well which would have resulted in loss of surging action and the screen might have been damaged. The top of gravel pack "A" was extended one-foot above the top of the valley alluvium to allow for settlement in the gravel pack and thus insure that the coarser gravel pack "B" did not come in contact with the valley sands.

A thick gravel pack, designated as gravel pack "B" on Fig. No. 5 was provided in the chalk berm portion of the well. This gravel pack does not act as a filter as the flow in the chalk portion of the well is out of the well screen and into the chalk berm. The selected gradation gave a gravel of fairly uniform sizes that would pass the flow from the well with a minimum head loss and was free of fine material that might choke the voids in the chalk.

The added thickness of gravel pack "B" was provided in order to (1) reduce the danger of bridging in the gravel pack as settlement of the pack occurred which would probably be increased by movement of filter material into the chalk berm, and (2) to protect the wood stave screen and riser pipe from the heat that may be generated by the oxidation and combustion of the fine disseminated iron pyrite in the chalk. Gravel pack "B" was extended to the surface to facilitate replacement of gravel lost by settlement.

Each well was fitted with a corrugated metal well pit and cover to protect the well and prevent caving of the chalk berm at the surface. The cover was designed to provide a tight fit to minimize drying out of the wood pipe. The well pits were extended two feet above the finished surface of the chalk berm and supported by compacting the chalk into the corrugations on the outer surface of the pipe. The pits were bottomless to permit settlement and replacement of the gravel pack.

Description of the Well System

The relief well system consists of 36 wells spaced at intervals of 70 and 100 feet. The wells were installed during 1953 and 1954. Construction of the downstream chalk berm had not yet been completed in the closure area, Station 61 + 50 to 75 + 00. Consequently the wells in that area were completed subsequently by extension up through the berm concurrent with construction of the remainder of the berm. Portions of the line of wells was offset to place the wells just beyond the downstream toe of the impervious embankment and in order to avoid surface obstruction. Fourteen (14) well point piezometers were installed along the line of wells to measure pressures in the foundation sands.

The line of wells was extended toward the right bank as far as the depth of bedrock permitted. A small amount of seepage was anticipated through the thin pervious layer overlying the bedrock beneath the right bank beyond the line of wells. The thickness of this stratum combined with the high elevation of the ground surface and the fact that seepage into the thin layer of pervious material was cut off upstream did not make practical the extension of the line

of wells further into the right bank. It was anticipated that the seepage from this thin layer would be towards the valley alluvium and would be at least partially intercepted by the end wells. The well spacing in the area from Station 61 + 50 to 65 + 50 was arbitrarily reduced from the computed value of 120 feet to 100 feet to provide for this additional flow. The remaining wells from station 65 + 50 to 75 + 00 were also placed in 100 foot centers.

The well spacing in the deep valley section from station 75 to 81 + 00 was 70 feet and that from station 81 to 85 + 00 was 80 feet. This reduced spacing was due to the increase in depth of the valley alluvium which increased the well discharge with a resultant increase in the head loss in flow out of the well screen into the chalk berm.

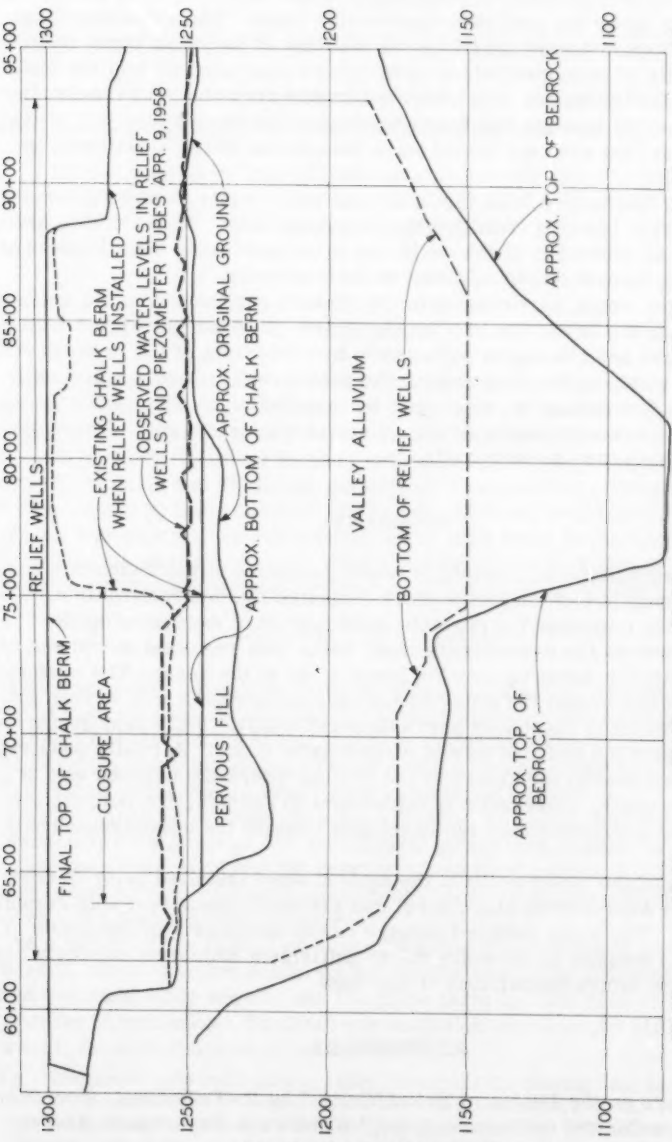
The left bank end of the line of relief wells was terminated at the switchyard. The chalk berm in this area was placed in relatively thin lifts, 3 feet as compared with 10 feet in the remainder of the chalk berm, and was somewhat compacted by the passage of hauling equipment. It was assumed that this portion of the chalk berm was therefore more impervious and might not satisfactorily conduct the flow away from the wells. The possibility also existed that flow through the chalk might cause settlement with resultant damage to the structure and electrical installations in the switchyard area. The discharge channel slope of the switchyard area was provided with riprap and a filter for a sufficient distance downstream to provide a means of controlled pressure relief.

Observations Subsequent to Construction

Periodic observations have been made of the piezometric pressures in the valley alluvium beneath the dam as measured by (1) the water levels in the lines of piezometers normal to the axis of the dam, (2) the water levels in the piezometers installed along the line of relief wells, and (3) the water levels in the relief wells. Observations have also been made of the discharge of the individual relief wells.

Piezometric water pressures in the valley alluvium beneath the dam indicate that fairly consistent gradients exist across the valley. Typical gradients for April 1958 are shown on Figs. No. 7, 8 and 9. Further, no excessive pressures are indicated in the foundation sands beneath the dam or beneath the downstream chalk berm.

A profile of typical water levels observed in the relief wells and intermediate piezometers is shown on Fig. No. 12. It is noted that the water levels in the relief wells in the closure area are consistently approximately 7 feet higher than the levels observed in the remainder of the wells. The piezometers in the closure area have generally indicated pressures approximately 7 feet lower than those observed at the wells as compared with those observed elsewhere along the line of wells. The cause of the higher water levels observed in the closure area relief wells and the difference between the levels observed in the wells and those in the intermediate piezometers has not been determined. Several factors however may have a bearing including: the absence of a natural blanket overlying the valley alluvium in the closure area, the substantial depth of relatively clean dredged sand fill placed in the river channel in this area, the fact that the lower portion of the chalk berm in the closure area consisted of more finely broken up material with less voids than elsewhere along the line of wells which could result in less flow out of the



PROFILE ON LINE OF RELIEF WELLS
SHOWING TYPICAL OBSERVED WATER LEVELS IN RELIEF WELLS AND INTERMEDIATE PIEZOMETERS

FIGURE NO 12

wells with a resultant build up of head in the wells, and the possible existence of two separate acquifers in this area with higher pressures existing in the lower acquifer which would be penetrated by the relatively deep relief wells but might not be by the shallower piezometer tubes. The existence of the relatively compact impervious layer at the base of the chalk berm reduces the possibility of movement of the underlying valley sand up into the chalk berm. To date the higher head observed in this area has not been detrimental. No excessive seepage has been observed at the downstream toe of the chalk berm in this area nor is any local subsidence of the chalk berm in evidence.

Observed discharges from the individual relief wells have been somewhat erratic but have shown a trend toward decreased flow. This trend is probably due to reduced efficiency of the wells and to an increasing effectiveness of the upstream blanket due to siltation in the reservoir.

Some of the wells, particularly in the closure section where the wells were installed when the berm was at a low level and extended concurrent with construction, have been damaged sufficiently by settlement of the berm to render them inaccessible to the flow water. Current investigations are aimed at checking the functioning of these wells by installation of additional piezometers as well as determination of the causes of the higher water levels observed in the closure section wells.

SUMMARY

Early observations of hydrostatic water pressure in the foundation sands beneath the dam at low reservoir pools indicated the desirability of a relief well system to minimize the possible development of excessive uplift pressures beneath the downstream chalk berm with resultant movement of the fine foundation sands up into the large voids in the berm. The system installed is unique in that the water is discharged through a well screen into the chalk berm which is relatively pervious horizontally. Fully penetrating wells were used when the bedrock was at a reasonable depth. Partially penetrating wells were utilized in the center of the system where the bedrock was at a considerable depth. Observations subsequent to construction indicate that excessive uplift pressures have not developed beneath the downstream chalk berm.

A portion of the relief wells in the system were installed prior to completion of the surrounding chalk berm and extended concurrent with completion of the berm. The wells suffered damage due to settlement of the berm. The possibility of damage to the wells due to settlement should be carefully considered in any future installation of this type.

REFERENCES

1. "Comments on the Design of Relief Wells," by P. T. Bennett, from "Conference on Control of Underseepage," Waterways Experiment Station, Vicksburg, Miss., 1945.
2. Discussion of Paper "Relief Well for Dams and Levees" by Turnbull and Mansur, Transaction ASCE Vol. 119, 1954, page 862.

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A REVIEW OF THE ENGINEERING CHARACTERISTICS OF PEAT

Ivan C. MacFarlane[1]

SUMMARY

This paper represents a resume of information contained in a number of references concerned in some way with physical and mechanical properties of peat. Information is presented under the headings: Classification; Per Cent Ash; Acidity Reaction; Density and Specific Gravity; Water Holding Capacity; Void Ratio and Shrinkage on Drying; Permeability; Shear Strength; Bearing Capacity; Consolidation Characteristics and Settlement.

From this report it is evident that many gaps exist in the knowledge of engineering properties of peat. What information is available is scanty, sometimes contradictory and confusing. A list of 25 references is included.

Little work has been done in determining the physical properties of peat. Most of the research carried out has been concerned with botanical and chemical characteristics of this complex material. The investigation of specific physical and mechanical characteristics of peat which could aid in the prediction of performance under load has scarcely begun.

Twenty-three references have been located, however, which are concerned in some way with physical and mechanical properties of peat and this paper represents a résumé of the information which they contain. This information is presented without comment even though there is some conflict of results and opinions between various papers.

As a result of the lack of co-ordination that has existed in muskeg and peat research, there is some confusion in terminology in the references. An attempt has been made here to use the same term in each case to describe any particular phenomenon. To avoid any confusion which might still exist, however, some definitions of terms follow:

Note: Discussion open until July 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1937 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 1, February, 1959.

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Per Cent Ash, Organic Content.—Ash content is the percentage of the total weight of ash or mineral residue remaining after a peat sample has been ignited. Organic content is the percentage of the total weight which is lost through ignition.

pH—is the measure of the acidity of a soil. It is defined as the negative logarithm of the hydrogen ion concentration in an aqueous suspension of the soil.

Mass Specific Gravity, Unit Weight, Volume Weight, Bulk Density, Wet Density—are all terms describing the same thing, namely the weight of a material (including solid particles and any contained water) per unit volume, including voids.

Dry Density—the weight of the dry material after drying to a constant weight at 105° C contained in a unit volume of moist material.

Apparent Specific Gravity—defined as the ratio of the weight of a given apparent volume of the material in a definite condition to the weight of the same volume of water. It is determined by using sand in a picnometer rather than water. The weight of sand replaced by the peat sample is converted to volume, which represents the apparent volume of the sample.

Absolute Specific Gravity (oven dry condition), Specific Gravity of Soil Solids—the ratio of the density of the soil solids to that of water.

Water Holding Capacity—Ability of a soil to take up and hold water.

Water Content, Moisture Content—in soil mechanics it is the loss in weight, expressed as a percentage of the dry material, when soil is dried to a constant weight at 105° C.

Also, in this report the term "muskeg" is used only when referring to the terrain whereas "peat" is used to describe the decomposed organic material.

In an effort to attain a certain degree of simplicity, this paper is completely descriptive; no graphs, tables or photographs have been included.

Classification

The peat samples tested are not classified or even described in detail in many of the references cited. Even when classified the samples are sometimes referred to only as "well humified" or "fibrous". Colley[2] differentiates between peat (65% - 100% organic matter) and muck (25% to 65% organic matter) and briefly describes the characteristics of both. Farnham[5] has developed a classification system of Minnesota peats and uses physical properties such as water-holding capacity, specific gravity, pH, etc., to identify the various classes. Feustel and Byers[6] give a detailed description of the peat samples they tested, based upon the botanical composition of the peat. Hanrahan[7,9] illustrates the cross-section of a muskeg area under study and refers to the peat as "fibrous" or "humified" but does not further designate the type of samples tested in shear and compression. Risi[16] describes in some detail the Quebec peat deposits investigated as to the plant species making up the deposit and those comprising the living surface cover. Smith[18] classifies British muskeg areas according to a topographical classification, namely, "fen peat", "raised bog", and "blanket bog". In each of these cases, the author uses his own descriptive system for the terrain and the material and there is no real correlation between the various references. This makes it almost impossible to compare the results of engineering tests in several references cited. Often, however, a range of values is given which covers most peat types from the very fibrous to the highly decomposed or amorphous peat.

Per Cent Ash

The proportion of organic matter in peat varies with the locality and with the types of samples tested. Peat which is mainly free of extraneous mineral matter has up to 95% organic content based on dry weight. Every gradation down to a pure mineral soil is found from this maximum. The organic material is generally combustible carbonaceous matter while the mineral constituent, whether part of the plant growth or extraneous matter, is incombustible and ash-forming. In general, the loss-on-ignition method of carbon-content determination serves as a satisfactory means of obtaining the percentages of organic and inorganic material. Colley[2] reports an average ash content of 17% for his samples. Farnham[5] gives figures for ash content which range from 4.5% for a moss peat to 66.6% for an amorphous peat. Risi[16] gives average mean values of ash content for each of the several Quebec bogs investigated; these values vary from 2.5% to 11.34%. He concludes that the ash content generally increases with depth. Shea[17] states that the average mineral or ash content of typical Everglades peat is less than 10%. Smith[18] refers to a variation in ash content from 2.0% for a cotton grass peat to 20.1% for a grass moor peat, although the latter was highly contaminated with mineral matter.

Acidity Reaction

Usually peat has an acid reaction, the so-called high moor peats being the most acidic. Low moor peats vary from strongly acid to neutral and slightly alkaline. Farnham[5] gives values of pH from 3.8 for moss peat to 7.5 for aquatic peat. Feustel and Byers[6] observed pH values ranging from 3.1 for heath peat to 7.1 for saw-grass peat. Risi[16] indicates that while the pH varies considerably, it appears to increase with depth (i.e., a decrease in acidity with depth). He gives values ranging from 4.8 to 5.6.

Density and Specific Gravity

The mass specific gravity (unit weight, volume weight or bulk density in the natural state) depends upon the moisture conditions of the sample. The specific gravity of the soil solids depends upon the organic and inorganic content of the sample. In most of the references cited the density or specific gravity of the peat is noted.

Colley[2] gives an average value for mass specific gravity of 0.95 (59.4 lb/cu ft) for wet peat and 0.20 (12.5 lb/cu ft) for dry peat. Farnham[5] gives average bulk densities for his different peat categories as follows:

moss peat	0.4 (25.0 lb/cu ft)
woody peat	0.6 (37.4 lb/cu ft)
herbaceous peat	0.7 (43.7 lb/cu ft)
aquatic peat	0.9 (56.1 lb/cu ft)
aggregate peat	1.1 (68.6 lb/cu ft)
amorphous peat	2.2 (74.9 lb/cu ft)

Feustel and Byers[6] show a range of volume weight of peat from 0.413 gm/cc to 1.158 gm/cc or 25.8 lb/cu ft to 72.2 lb/cu ft. Hanrahan[7,9] gives a range of density from 59 lb/cu ft to 64 lb/cu ft for wet peat and 4 lb/cu ft to 8 lb/cu ft for dry peat. Lewis[14] noted a bulk density of 67 lb/cu ft for the peat with which he was concerned. Shea[17] observed a dry unit weight of 5 to 10 lb/cu ft while Smith[18] shows a range for dry density of 4.7 to 19.3 lb/cu ft although

this last value represents a peat quite highly charged with mineral matter. Thompson and Palmer[19] report a unit weight range of 59.6 to 70.7 lb/cu ft for wet peat and 14.5 to 20.8 lb/cu ft for dry peat.

Observations of specific gravity of the soil solids reveal a gratifying consistency in the various references. Cook[3] shows a range of specific gravity of 1.85 to 2.45, the latter value representing peat highly charged with mineral matter, and notes that the water content increases as the specific gravity decreases. Feustel and Byers[6] give values for apparent specific gravity and absolute specific gravity for various peat types. The apparent specific gravity is the ratio of the weight of a given apparent volume of the material in a definite condition to the weight of the same volume of water. The values for apparent specific gravity in the natural state correspond to those for unit weight in the metric system. These values lie somewhere between those for apparent specific gravity in the oven-dry condition and in the saturated condition. The apparent specific gravity in the oven-dry condition is rather difficult to determine. The results depend upon the porosity of the peat and range from as low as 0.06 for some moss peats to as high as 1.21 for some sedimentary or more compact peats, although most of the results recorded by Feustel and Byers give values between 0.3 and 0.9. The apparent specific gravity in the saturated condition always lies somewhere between unity and the value for the specific gravity of the soil solids. Average values of apparent specific gravity for different type peats are given by Feustel and Byers as:

1.016 for sphagnum peat
1.070 for heath peat
1.108 for Everglades peat
1.059 for sedge peat.

The absolute specific gravity in the oven-dry condition (specific gravity of the soil solids) of Feustel and Byers' samples varied from 1.105 to 2.161 with the average values for the different types of peat being as follows:

1.510 for sphagnum peat
1.405 for heath peat
1.637 for Everglades peat
1.505 for sedge peat.

In his papers, Hanrahan[7,8,9] reports a range of specific gravity for the soil solids of 1.1 to 1.8. Hardy and Thomson[11] give a specific gravity value of 1.4. Ranges of 1.795 to 2.036 and 1.2 to 1.5 are quoted by Thompson and Palmer[19] and Ward[21] respectively. The higher values (over 2.0) indicate a considerable degree of mineral contamination.

Water-Holding Capacity

Peat has a great capacity for taking up and holding water. This affinity for water is one of the most important characteristics of the material. It is significant that most of the differences in physical characteristics are due, at least in part, to the moisture content. The ability of peat to absorb and hold water has been ascribed to five different phenomena.[24] These are (1) water of occlusion, held in pores 1 millimetre or more in diameter; (2) capillary water; (3) colloidally bound or absorbed water; (4) osmotically bound water; and (5) chemically combined water. The maximum moisture holding capacity

represents a total of all forms. The quantity of water held in peat varies considerably, being less in the more decomposed (amorphous) types than in the more fibrous types. The moisture content also varies with the season and with the corresponding position of the groundwater table. In its ability to soak up water, peat acts like a sponge. In its natural condition, peat generally contains sufficient water to saturate the vegetable matter. Most of the references cited refer to the exceedingly high water content which constitutes up to 95% of the total weight of the mixture. The highest value is recorded by Feustel and Byers[6] for a sphagnum peat which had a moisture content of 3235%, calculated as a percentage of the weight of the dry peat. Generally, the range of moisture contents appears to be between 750% and 1500%. Although moisture contents as low as 150% have been noted,[3,18] these undoubtedly represent peats with a high degree of mineral contamination.

When peat is removed from its source of moisture, and allowed to become air-dry, the moisture will dry to a very low value in most cases. Feustel and Byers[6] note a range of moisture content for air-dried samples of 4.2 to 12% of the dry weight of the peat. Once peat has been air-dried, it loses some of its moisture-holding capacity and is slow becoming saturated again. The peat will not re-absorb as much water as it originally held. According to the tests carried out by Feustel and Byers, the average values for the different peat types considered are as follows, (the figures denoting the percentage of the water capable of being held which is taken up again after air-drying):

Sphagnum peats	55%
Heath peats	33%
Everglades peat	40%
Sedge peat	52%

In draining water from peat, only a part of the moisture may be removed by pressure. This is probably the water of occlusion. The remaining water resists great pressure; application of heat and subsequent evaporation are the only possible means of drying. The moisture equivalent is a means of measuring this power to retain water in spite of pressure. In the centrifuge moisture tests, separation of soil and water is attempted by using a centrifugal acceleration of 1000 times the force of gravity. The centrifuge moisture equivalent (C.M.E.) is stated as the weight of the water retained, expressed as a percentage of the dry weight of the soil. Feustel and Byers carried out this test on natural peat samples and on samples which had been air-dried and resaturated. The moisture equivalent is less for the previously air-dried sample than for the natural sample. However, the difference between the two values is less for peat near the surface than from deeper layers. The moisture equivalent is considerably less for highly decomposed (amorphous) peat than for fibrous peat. The moisture equivalent of natural samples tested by Feustel and Byers varies from 91% for well-decomposed peat samples to 628% for fibrous peat samples. For air-dried and resaturated samples, the range was from 46% to 600%.

Void Ratio and Shrinkage on Drying

The void ratio is the ratio of the volume of voids to the volume of soil solids and, for a fully saturated soil, is equal to the product of the moisture content and the specific gravity of the soil solids divided by 100. As an

example, consider a typical peat sample with a moisture content of 1000% in the saturated condition. The specific gravity of the soil solids may be taken as 1.6. The void ratio would be: 1000 x 1.6/100 = 16. A fibrous peat may have a very high void ratio, whereas that of an amorphous peat may be relatively low.

Colley[2] reports a range of void ratios from 4.6 to 10.3. Cook[3] gives a range of 2.84 to 13.08. Hanrahan[7,9] found that the void ratio varied from as low as 9 to as high as 25, the latter value being for a very fibrous peat. Such values indicate the settlement which might be expected for this type of peat. Thompson and Palmer[19] observed values of 5.1 to 7.09 for void ratio, whereas Ward[21] notes a range of 6 to 17 for void ratio of peat in its natural condition.

The void ratio is reduced in proportion to reduction in moisture content for a fully saturated peat. When the voids are filled with both water and air, however, the volume cannot be reduced beyond a fixed point by drainage alone. Therefore, as peat dries out it will shrink, becoming harder and more firm. The greater the water reduction the more the shrinkage, up to a particular point. The surface of a muskeg area may develop wide shrinkage cracks due to drying, which might occur under drought conditions or because of drainage.

Colley[2] found from drying out several peat samples that the shrinkage of a sample varies from 50% to 10% of the original volume. Feustel and Byers[6] calculated the maximum percentage of shrinkage possible from the values for maximum moisture-holding capacity and the apparent specific gravity in the oven-dry condition. The following example demonstrates the method of calculation:

Previously determined by test:

Maximum moisture-holding capacity = 378%
Apparent specific gravity (saturated) = 1.162
Apparent specific gravity (oven-dry) = 0.41
Absolute specific gravity = 1.548

$$\text{Void ratio} = \frac{\text{Absolute specific gravity x moisture content}}{100}$$

$$= \frac{1.548 \times 378}{100} = 5.85$$

$$\text{Porosity} = \frac{5.85}{5.85 + 1} = 85.4\% \text{ of total volume.}$$

Weight of oven-dry material in 1 cubic centimetre of saturated peat:

$$= \frac{1.162}{\frac{378 + 100}{100}} = \frac{1.162}{4.78} = 0.243$$

Apparent volume of 1 gram of oven-dry peat in the saturated condition:

$$= \frac{1}{0.243} = 4.12 \text{ cc}$$

Apparent volume of 1 gram of oven-dry peat:

$$= \frac{1}{0.41} = 2.44 \text{ cc}$$

$$\text{Maximum shrinkage} = \frac{4.12 - 2.44}{4.12} \times 100 = 41\%$$

The results are only approximate since the apparent specific gravity determinations of the oven-dry material are themselves only approximate, yet they serve as a fair basis for comparison. Results obtained for various types of peat from widespread localities in the U. S. range from 23% to 90% maximum shrinkage.

Hanrahan[7] gives results for linear-shrinkage and volumetric-shrinkage tests on peat. He attempted to correlate shrinkage with various properties of peat, such as degree of humification, compressibility and dry density. However, it was difficult to make these correlations. It is pointed out that shrinkage forces in peats can be extremely high.

Permeability

Permeability may be defined as the property of a substance which permits the passage of fluids through the pores of the material. Colley,[2] in a series of preliminary experiments measured the permeability of undisturbed peat samples by using both the variable and constant-head permeameter. The average value obtained for the permeability in the vertical direction was 0.3 ft per day. The values for the permeability in the horizontal direction were erratic, ranging from 0.2 to 1.3 ft per day.

Cuperus[4] devised a special apparatus for measuring the permeability of peat in the field. Experiments were carried out to determine the reasons for failure of a sand fill over soft peat. Measurements were taken of the water permeability of the peat where failure of a test embankment occurred and also in sections where there was no failure. Cuperus concluded that the difference in the behaviour of the peat resulted from a difference in the permeability. The peat in the section which failed had a lower permeability than that in the sections which did not fail.

Hanrahan[7,9] determined the permeability of peat in a falling-head permeameter and in the consolidation cell. He found that the permeability is affected considerably by the magnitude and duration of loading. The following results from a test on a specimen of partly humified peat taken from the vicinity of a drainage trench indicate the order of the variation which takes place:

Before test—
(void ratio = 12) $K_w = 4 \times 10^{-4}$ cm/sec.
After 2 days under load of 8 psi—
(void ratio = 6.75) $K_w = 2 \times 10^{-6}$ cm/sec.
After 7 months under load of 8 psi—
(void ratio = 4.50) $K_w = 8 \times 10^{-9}$ cm/sec.

It is seen, therefore, that after 7 months the peat thus loaded is 50,000 times less permeable than it was initially.

Strength and Deformation Characteristics

Muskeg possesses some resistance to compression and certain types can carry quite considerable loads providing point loading is not sufficient to cause a rupture of the surface. It appears that the strength of a muskeg area would depend upon the type and depth of the surface vegetative mat, the type and depth of the peat and the conditions of moisture.

Most of the engineering problems with muskeg would generally fall into the following three categories:

1. Shear strength of the peat
2. Bearing capacity
3. Settlement characteristics

Several of the references cited are concerned with one or more of these aspects.

Shear Strength

There are three methods of assessing the shearing strength of peat:

1. Measuring the strength in situ by means of some apparatus such as the vane tester
2. Using the stability analysis in an area where a sliding failure has occurred
3. Securing undisturbed samples in the field and performing laboratory shear tests

Vane tests in peat have been carried out by Hardy and Thomson[11] and Smith.[18] Hardy and Thomson did some field vane tests near mile 253 on the Alaska Highway. The depth of the muskeg varied from 12 to 17 feet and was underlain by a soft blue clay. The peat was distinctly fibrous at all depths although the degree of decomposition of the organic material increased with depth. Test results show that shearing strength increases directly with depth although the maximum shearing strength was only developed after an extraordinary high degree of deformation. A typical set of shearing-strength determinations on one of the test holes shows a range of values from 106 lb/sq ft at 1 ft 9-in. depth to 610 lb/sq ft at 12-ft depth.

Smith measured the shearing strengths of the near-surface layers of some British muskeg areas in connection with trafficability studies. He concluded that all peats which have not dried out and which retain elements of their original vegetation have strength profiles of a similar form. For moss peats, the strength profile exhibits a linear decrease in strength with depth from a shearing resistance of 2-1/2 psi (360 lb/sq ft) at 4-in. depth to one of about 1/2 psi (72 lb/sq ft) at 22-in. depth. For fen peats, these values should be increased by 1 psi at all depths. Smith points out that mineral material can increase the strength of peat if present in amounts greater than 20%. The effect of mineral contamination, drying out and development of pasture can result in an increase in strength of the surface layers from 50 to 150%. Finally, he states that the moisture content of the peat can give no guide to its strength unless the degree of humification and amount of mineral contamination, if any, are known.

Hardy[10] and Hardy and Thomson[12] are the only two references which mention the use of a stability analysis of a sliding failure as a means of determining the shearing strength of peat. In the analysis of one such failure, the average shearing strength was found to be of the order of 82 to 85 lb/sq ft. The analysis of the shearing stresses involved was made on the assumption that the shearing failure occurred on cylindrical surfaces intersecting the ground surface at the toes of the disturbed areas on each side of the road. Working back, it is possible to predict the height of embankment which would cause a shear failure in the peat. Hardy and Thomson state that in comparing the measured shearing strengths with the rather scant results available for shearing strengths of peat based on slide analyses, the vane tests show higher strengths at depth than would be expected from the stability analyses. On the other hand, limited unconfined compression tests that were run on undisturbed

samples of the higher strength material encountered in the test holes gave shearing strengths ranging from 150 lb/sq ft to as much as 430 lb/sq ft, with a fairly good correlation with the results for the vane test at the same depth. The shearing strengths from the vane tests at shallow depths are in fairly close agreement with the available data from stability analyses.

Turning to laboratory shear strength tests, Hanrahan[7,8,9] in his pioneer work carried out unconfined compression tests and triaxial tests on peat samples. For peat not previously loaded, the shear strength from unconfined compression tests was found to range between zero for undrained peat to about 4 psi (576 lb/sq ft) for drained peat of about 7 lb/cu ft dry density. Average values of drained peat were 2 to 4 psi (288 to 576 lb/sq ft), the strength being related to a deformation of up to 20% of the initial length of the sample. The use of the triaxial compression apparatus for the investigation of shearing strength is described. Results are given of triaxial tests during which measurements were made of pore-water pressure. It is concluded that in both the normally loaded and preconsolidated states the shearing strength of peat depends primarily on the water content according to an established relationship.

Jankowski[13] states that the angle of "internal friction" in peats decreases with an increase in the degree of decomposition, making water content unimportant at a degree of decomposition not exceeding 50%. Not until the degree of decomposition exceeds 50% does the angle of "internal friction" decrease sharply.

Shea[17] carried out direct shear and triaxial tests on peat samples. He states that direct shear tests give completely erroneous results. The predetermined failure plane cuts across the fibres and the apparent angle of internal friction is usually between 20 and 25 degrees. When the same material was tested in a confined compression apparatus, where failure can occur on any plane, a very low shearing strength was obtained. From triaxial tests, the angle of internal friction was found to be usually less than 5 degrees. The only shearing strength considered in design by Shea was cohesion of about 0.1 ton/sq ft.

Ward[21] ran a series of unconfined compression tests on peat to quickly determine the shearing strength for the analysis of a slip of a flood defence bank constructed on muskeg. The mean unconfined compressive strength (excluding the upper crust) in the vicinity of the slip was 1.75 psi (range of 0.56 to 2.61 psi) or 258 lb/sq ft from other parts of the bog, the mean strength was 1.72 psi (range of 0.50 to 2.78 psi) or 248 lb/sq ft. Appreciable quantities of water were squeezed out during the tests but were included in the water content determinations. Ward observed that an appreciable change in the water content of this peat had little effect on its strength, the variation in strength being largely due to the different plant fibre structures and to the degree of humification.

Ward, Penman and Gibson[22] performed a series of drained triaxial tests and equilibrium shear box tests on some peat samples. Each specimen was consolidated for about a week. The results indicated that $c' = 100$ lb/sq ft and $\phi' = 18$ degrees for the range of principal stresses involved on the slip under investigation. The shear resistance of the peat under the overburden at the start of construction was about 270 lb/sq ft although the great variability of the peat caused uncertainty as to how typical were these test results.

Bearing Capacity

Only two of the references refer in any way to the bearing capacity of muskeg. Smith[18] develops a simple formula for translating the measured values of shearing resistance into effective bearing capacity at the surface of the muskeg by considering that the peat behaves as a purely elastic medium until plastic flow begins. The bearing capacity is stated to be equal to π times the shearing resistance when the plastic condition is first reached. The ultimate bearing capacity is arbitrarily assumed to be twice this value. He estimates that the peats examined average an ultimate surface bearing capacity of about 10 psi (1440 lb/sq ft). The average ground pressure should be considerably less than this figure because of the high peak stresses set up under a wheel or track. From limited records, Smith suggests that this pressure should be about 3 psi if a vehicle is to cross the terrain easily.

In a translation from the Russian[23] it is pointed out that the bearing capacity of the so-called "low bogs" depends upon the dimensions and shape of the supporting surface and may be calculated from the formula $q = A_0 + V_0(P/S)$, where A_0 and V_0 are constants characterizing the strength of the peat and not dependent on the dimensions and shape of the supporting area, P is the perimeter in cm and S the area in sq cm. The author states that the bearing capacity of sand increases with an increase in the dimensions of the supporting area, but on muskeg it decreases. For loading surfaces of a constant area, the value of the bearing capacity of muskeg depends upon the shape of the area. When the area is increased, "q" decreases and tends towards a certain value characteristic for each peat deposit. Change in the value of "q" is especially great for bearing areas of small dimensions.

Consolidation Characteristics and Settlement

Long-duration consolidation tests on peat were carried out by Buisman[1] as early as 1936. Results of laboratory tests of short and long duration on peat and clay samples are plotted on a semi-log scale. From these tests—one of which lasted 500 days—he concludes that for peat samples of 2 cm thickness, the log time - settlement curve becomes a straight line approximately a minute after the load is added and remains a straight line throughout the observations. He noted that the log t - s diagram for a road embankment on peat (observed over a period of 2 years) was also a straight line.

Colley[2] carried out consolidation tests with the conventional type consolidometer and with the triaxial apparatus. With the latter a scale-size sand drain was installed in the centre of the sample to observe its effect on the rate of consolidation. He reports that a typical sample when consolidated in this manner increased the rate of consolidation approximately 12 times. Time - settlement curves are shown for typical samples, one curve for the consolidation occurring during a conventional test on peat and another showing the acceleration of settlement due to the addition of vertical sand drains. It is noted that the same amount of settlement which occurs in a 6-month period with sand drains requires 18 months to complete when sand drains are not used.

Cook[3] gives a graph showing the relationship between coefficient of compressibility and water content of unconsolidated peat (i.e., no preloading). The coefficient of compressibility decreases in a straight line relationship with a decrease in the water content.

Hanrahan[7,8,9] investigated the compressibility and rate of consolidation of peat. He found that compressibility of peat decreases with increasing load

and that for a given load, the greater the dry density the less the settlement. Consolidation tests were performed on undisturbed and remoulded samples and on specimens of different thickness. He notes that the curve of settlement plotted against time for peat is quite different from the curve setting out the solution of the basic differential equation of consolidation. He ascribed the causes of the discrepancy between the two curves to a variety of phenomena including the abnormally large decrease in permeability which accompanies the application of load, the decreasing coefficient of compressibility and thixotropy. The surface activity and adsorption properties of peat are also high, so that considerable secondary consolidation may be expected. Hanrahan concludes that for peat layers of varying thicknesses the magnitude and time of settlement depend upon the ratios of the thickness and the square of the ratio of the thickness, respectively, for a considerable period after the completion of the primary consolidation phase.

Lewis[14] presents the results of an investigation into the approach embankments to a new bridge. Since the underlying soil contained a 3-1/2-ft layer of very compressible peat, appreciable settlement of the embankment was anticipated. Three settlement gauges were installed at the site before the embankment was constructed. Laboratory consolidation tests were carried out on peat samples prior to construction in order to estimate the amount of settlement likely to occur. Since the consolidation for each load increment was considerable even after the usual 24-hr period, long-term consolidation tests were carried out. The load increments were allowed to act until the rate of secondary consolidation had decreased to a very small value. The time required to reach this condition was about 50 days. When, for the first 24-hr period, compression was plotted against square root of time the curve was linear over only about the first two minutes. In theory this corresponds to about 60% of the primary consolidation, and Lewis suggests that 100% primary consolidation was almost completed within the first 10 minutes. As much of the consolidation of the laboratory test samples was of a secondary nature, it was not possible to calculate settlement-time curves for the embankment on the basis of the classical consolidation theory. However, the results of the long-term consolidation tests together with data obtained at the site (thickness and bulk density of the compressible layers) were used to estimate the ultimate settlement likely to occur at the three sites where settlement observations were made. Lewis gives the following values for calculated and observed settlements, the latter having been made over a period of about 4 years:

	Settlement (inches)			
	Gauge 1	Gauge 2	Gauge 3	Avg.
Calculated on basis of long-term consolidation tests	10.1	9.0	9.2	9.4
Calculated on basis of tests with 24-hr loading periods	8.1	6.7	7.2	7.3
Observed settlements (up to July/55)	10.2	10.4	7.2	9.3

It was noted that the calculated settlements do not take into account the effect of traffic.

Ringeling[15] investigated the settlement characteristics of a peat layer 2 to 3 metres thick beneath a projected road. It was necessary to determine

(1) how fast the compressible peat layer would settle under the applied embankment load, and (2) when the consolidation would be complete. Consolidation tests were run on several samples and an experimental road section was constructed to check the actual settlements against those computed from the laboratory tests. The observed settlements did not altogether agree with the calculated settlements, being less for lower loads and greater for the higher loads. Ringeling also constructed a device for measuring the pore-water pressure of peat in the field. He observed a fast increase in pore-water pressure with increased load and a subsequent slow decrease in pressure. Curves show that immediately after loading of the peat, about 75% of the load is taken by the pore water.

In the construction of levees on peat in the Everglades of Florida, Shea[17] pointed out that from the very beginning of the project it was recognized that large settlements of the embankments would be inevitable. Early estimates, based on previous construction experience and on very incomplete consolidation test data, were that settlement would be equal to 50% of the peat thickness and that half of the settlement would occur during construction. Before the placing of the fill, settlement plates were installed at many points along the levee alignments. The thickness of the peat at each plate was determined by auger borings. Elevations of the plates were checked several times after completion of the embankments. The average of all readings came very close to the original estimate of 50% of the peat thickness. Apparently, almost 100% of the settlement occurred during construction as there was very little change in the elevation of the plates after the first reading. Use of consolidation tests for the prediction of settlement was only partially successful. While the amount of settlement could be determined with reasonable accuracy, the predicted length of time required for settlement was not even approximately correct. The consolidation tests indicated 15 to 20 years for 90% settlement, whereas the actual settlement took place in a few days or weeks. Shea did not investigate this discrepancy further.

Thompson and Palmer[19] carried out consolidation tests on several samples of peat in order to determine the magnitude and rate of settlement of two earth-filled concrete barricades built on a tidal marsh underlain by nearly 50 ft of peat. They found that the thickness versus log-time curves were similar for all samples, curved for the first minute, a straight line from one minute to approximately one day, and then changing to another and steeper straight line for one day until the end of the tests (11, 20 and 36 days). They observed no similarity between these curves and those obtained for clay. There was no line of demarcation between primary and secondary consolidation in these tests. The primary consolidation was apparently completed in less than one minute. It was predicted that the plot of the settlement of the barricades versus log-time would be a straight line, in keeping with the results of the long-term consolidation tests. Actual observations of the settlements over a period of about 4 years confirmed this prediction.

Van Mierlo and Den Breeje[20] ran a number of consolidation tests to predict the settlement of fills built on peat. They theorized that despite the numerous approximations and assumptions, it seems likely that the actual time - settlement curves would closely approximate the calculated ones. The rate of "secular" settlement after the primary phase is mainly based on compression constants found from the compression tests. However, the corresponding calculations had been checked by observations over a period of only a few years. They point out that it is necessary to make prolonged observations in areas of weak subsoil before it is possible to detect any substantial discrepancies between theory and fact.

Ward[21] noted that the peat he was investigating appeared to be the weakest and most compressible material of its type ever encountered in soil mechanics studies in Great Britain. The upper 18 inches was subject to drying and exhibited considerable tensile strength, but owing to the compressibility of the underlying peat the load of the banks produced an appreciable surface curvature. In several places tension cracks were observed on one side or other of the banks. One bank settled 4 ft in 2 years. It was necessary to build the banks (constructed of peat) 75% higher than specified to allow for subsidence caused by (a) settlement of underlying peat, and (b) drying shrinkage of the bank peat. As the banks dried out, the contraction produced cracks 1 to 2 in. wide which required maintenance.

CONCLUSIONS

While there has been gradual accumulation of useful information on the physical and mechanical properties of peat as evidenced by this report, it is obvious that many gaps in the knowledge exist. This is especially true with regard to the strength characteristics of peat, where the information is particularly scanty, sometimes contradictory and confusing. Furthermore, it would be most desirable if a consistent system for classification of peat could be used, so that test results of various workers in this field could be more intelligently compared. Attempting to correlate the characteristics of fibrous and amorphous peats can be compared roughly to attempting to correlate the properties of a sand and a clay. In Canada a classification system for muskeg and peat has been devised[25] and is now gradually being accepted so that terminology is being standardized. This is only the first step in the solution of the many problems presented by this unusual type of terrain.

This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the Director of the Division.

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Note: Paper 1951 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, SM 1, February, 1959.

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A METHOD TO DESCRIBE SOIL TEMPERATURE VARIATION[a]

Closure by E. B. Penrod, W. W. Walton, and D. V. Terrell

E. B. PENROD,[1] W. W. WALTON,[2] and D. V. TERRELL,[3] M. ASCE.—
The authors were pleased to learn from Mr. Pearce that the analysis of ground-temperature data at Ottawa and Saskatoon, for the years 1952 to 1955 has been reported. It is gratifying to learn that the results reported by the National Research Council of Canada are similar to those reported for Lexington, Kentucky. It is of interest to know that the parameter $\frac{P}{\pi\alpha}$ remained relatively constant from year to year for both sand and clay for the two localities in Canada.

At Lexington, Kentucky soil temperatures have been recorded at two-hour intervals throughout the year since 1952 at depths of 0, 2, 4, 6, 8, and 10 ft. Averaged values of the soil temperatures at each depth were obtained for each month of 1952. In developing equations (14) and (15), only soil temperatures at depths of 4, 6, 8, and 10 ft. were considered, thereby eliminating the effects of erratic changes of temperature near the earth's surface. Research in geophysics is still in progress at the University of Kentucky. At the conclusion of this project, an equation similar to (14) will be developed, based on average values of temperature for the period considered.

ERRATA

Page 1537-4, Eq. (11) should read $\Delta T = \frac{\Delta x}{2}\sqrt{\frac{\pi}{\alpha P}}$

Page 1537-4, Eq. (12) should read $\alpha = \left(\frac{12}{2}\right)^2\left(\frac{\Delta x}{\Delta T}\right)^2 - \frac{1}{\pi P} = \frac{36}{\pi P\left(\frac{\Delta T}{\Delta x}\right)^2}$

Page 1537-6, near middle of the page should read --- Meeting in Montreal (6) ----- instead of (4).

Page 1537-6, line from bottom, change 51.1° to 5.1°.

Page 1537-10, change (Apr. 15 + 2.45) = 2.24 = Apr. 15 + 0.21 months to read (Apr. 15 + 2.45 - 2.24) = Apr. 15 + 0.21 months.

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a. Proc. Paper 1537, February, 1958, by E. B. Penrod, W. W. Walton, and D. V. Terrell.

1. Prof. and Head of Dept. of Mechanical Eng., Univ. of Kentucky, Lexington, Ky.
2. Associate Prof. of Mechanical Eng., Univ. of Kentucky, Lexington, Ky.
3. Dean Emeritus of the College of Eng., Univ. of Kentucky, Lexington, Ky.

Page 1537-10, middle of page change $\alpha_2 = \frac{36}{8766\ (0.224)^2}$ to read

read $\alpha_2 = \frac{36}{8866\ \pi\ (0.224)^2} = 0.0261\ ft^2/hr.$

Top of page 1537-11 change by adding α to read $-x\sqrt{\frac{\pi}{\alpha_2 P}}$ above both columns. Also note that -x is out of line.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: FRENCH GROUTING PRACTICE[a]

Closure by Armand Mayer

ARMAND MAYER,[1] M. ASCE.—Mr. Clark has raised three points in his discussion of the writer's paper.

1. First is why grouting of pervious soils has been more widely practiced in Europe. The matter has already been discussed with different American experts, and one of them said he did not know why these methods which never worked when tried in the laboratory in the U. S., did work when used in the field in France. The answer is there: tests in the laboratory are made on homogenous granular materials, tests in the field in layered, heterogenous deposits. The grout penetrates the pervious strata, the resultant permeability being small. It would in many cases not have penetrated remoulded samples from the site, tested in the laboratory, where the natural voids would have been already filled with fines.

2. Mr. Clark thinks there has been a mistake in the dimensions given for the Genissiat cut off. The figure given of 300 m^2 = 3000 sq feet is correct. The canyon was very narrow and the curtain short. Three lines of holes were grouted so that the amount of chemicals used was large. But it was the first time. Since then many other curtains have been made. The Serre-Ponçon curtain is around 3000 m^2 = 30.000 sq feet. The cut offs around the excavations in the Rhine valley are much larger still.

3. Mr. Clark does not want to let unchallenged the statement that high speed mixers can change the size distribution of cement grains. The writer is afraid that Mr. Clark has had little or no experience with high-velocity mixers.
   The French Cement Research Center has made very thorough studies on high velocity mixing The abrasive action the writer mentioned in his paper can be felt on the grains under 20 μ in size. The percentage of grains 5 μ in size is twice as large with high speed than with ordinary mixers. The same is true with all colloidal elements which accounts for the fact that the suspencions obtained with high speed mixed cement are much more stable than others.

---

a. Proc. Paper 1550, February, 1958, by Armand Mayer.
1. Cons. Engr., Chmn. of the French Cement Research Organization, Paris, France.

THE STRUCTURE OF COMPACTED CLAY[a]

Discussion by D. Hugh Trollope

D. HUGH TROLLOPE,[1] A. M. ASCE.—In presenting this paper Professor Lambe is to be congratulated on making a most significant contribution to the understanding of engineering soil behaviour. In many fields of engineering study there is frequently just cause for complaint at the time lag between discovery in one branch of scientific work and its subsequent application in associated fields. It is a tribute to Professor Lambe and his colleagues that recent advances in the field of colloid chemistry have already found application in soil mechanics studies.

There can be no doubt that the inter-colloid electrical forces are most important in determining the arrangement and subsequent behaviour of clay structures. In particular the author's emphasis on the dominating effect of environmental conditions during deposition or formation of the structure is to be endorsed. At the present time it appears to be reasonable to assume that, arising from the electrical nature of colloid activity, either a net repulsive or a net attractive force is developed between proximate clay particles. The author has identified this net interparticle force with the salt content of the pore fluid and has suggested the terms salt-flocculation and non-salt-flocculation to describe the resulting arrangements. However the theoretical background on which this is based derives from a study of dilute colloid suspensions. In engineering soils we are mostly concerned with much denser suspensions and it seems likely that other factors will contribute to structural arrangement. In a recent study(1) the writer has attempted to show that external forces contribute markedly to the nature of structurally possible clay particle arrangements, and also as the author recognises, a moisture deficit can produce equivalent 'salt' conditions in a soil without altering the total salt content. It is for these reasons that the writer feels that the terms salt and non-salt flocculation may be too restrictive in the descriptive sense. From an analysis of two-dimensional models(1) the writer would suggest as alternatives the relevant geometric terms-triangular for salt-flocculation and rhombic for non-salt-flocculation. Even these terms avoid the complications of three dimensional geometry so that tetrahedral and rhombohedral respectively may be more suitable and form a useful link with established crystal geometry.

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1. Reader in Civ. Eng., Univ. of Melbourne, Melbourne, Australia.

THE ENGINEERING BEHAVIOR OF COMPACTED CLAY[a]

Discussions by José A. Jiménez Salas and D. Hugh Trollope

JOSÉ A. JIMÉNEZ SALAS.[1]—As the General Reporter of the First Division of the IV International Conference on Soil Mechanics indicated (ROSENQUIST, 1957), a brief, but quite important monograph (VERWEY and OVERBEEK, 1948) on colloid chemistry, produced a considerable impact on the field of Soil Mechanics.

The writer of this discussion was partly responsible for one of the shock waves following this impact (SALAS and SERRATOSA, 1953), which was very soon followed by another wave, much more significant, by Mr. Lambe (1953).

Since then, several eminent researchers, including Bolt (1956), have dealt with the mechanical properties of clay, proving the feasibility and fertility of this kind of approach, in spite of some pessimistic predictions.

On account of the time elapsed, the need was felt of a publication gathering the work done during these last years, and establishing the basis of a new doctrine. The writer interprets this to be the aim of the current paper publication of Mr. Lambe.

Undoubtedly, this publication is to be studied and referred to in the future by those desiring to investigate along this line of thought. It will also be considered as representative of same in many discussions. It is therefore of importance to clear this publication of all cloudiness and of all doubtful assertions. This is the purpose of the writer's comments, which will mainly deal with the shear strength theory a vanced in Proc. Paper 1654. In the writer's opinion, a correct physicochemical approach of the problem is still missing, and undoubtedly will be quite different from that presented by Mr. Lambe.

Let us briefly consider the energy-distance curves for two adjacent micelles (Fig. 11 of Mr. Lambe's paper). When these two micelles are infinitely far apart, the interacting forces are nil. If we progressively draw the particles toward each other, we must supply work (positive or negative), which is stored in the form of free or potential energy.

In the first place, let us consider the case of dispersed particles (case a, pg. 16 of Mr. Lambe's paper). When the particles draw near, it is necessary to overcome the electrical repulsion R. The work to be supplied is positive, and the free energy accumulated is positive too. It increases in accordance with a more or less exponential law (VERWEY and OVERBEEK, 1948, pg. 95) and ends with a finite value for zero distance. This finite value is the free energy of the double layer of the system constituted by the two particles, joined in intimate contact.

On the contrary, when we are draw the particles together we are aided by

a. Proc. Paper 1655, May, 1958, by T. W. Lambe.

1. Prof. of Soil Mechanics and Foundations at the "Escuela Especial de Ingenieros de Caminos, Canales y Puertos," Madrid, Spain.

the attractive van der Waals' forces. These forces produce a work or, in other words, we must supply a negative work. This quantity of work increases more or less quadratically with decreasing distance, and in the limit it tends to $-\infty$ (VERWEY and OVERBEEK, 1948, pg. 101).

If the particles are let alone, they will automatically adopt an equilibrium position, which must be one of minimum potential. Neither curve 1 nor curve 2 in Fig. 11 of Mr. Lambe's paper do show a potential minimum, so that there cannot be any equilibrium position. In the system represented by curve 1, the particles will spontaneously draw near, releasing the greatest possible part of their free energy, until they reach close contact, and the system will not be dispersed any longer. The considerations made in the author's paper about the effects of pre-compression are not justified.

In the case of curve 2, we can assume that particles are separated by a distance smaller than $\underline{x}$. The particles will likewise tend to reach close contact. But in the event that the distance is greater than $\underline{x}$, the particles will tend to separate more and more. This is the case of a colloidal suspension. Point $\underline{x}$ represents an "energy barrier" that the particles cannot tresspass without the aid of an external force.

A particular case is that corresponding to distance $\underline{x}$. The derivative of potential with respect to distance is zero, that is to say, the attractive and repulsive forces are equal. This is an equilibrium position, but one of perfect instability. A movement in any sense will release energy. This is an impossible position, because it corresponds to a maximum of potential. On the contrary, if there were a minimum of potential, it would be necessary to supply work to displace the particle from its equilibrium position, and this would be stable.

However, there are two factors which affect the phenomenon, and which are not directly reflected on Fig. 11. First: the laws of attraction and repulsion change for very small distances between particles; steric interaction I, as it has been called in Mr. Lambe's paper, makes its appearance. Second: in many cases, there is a potential minimum, a shallow one, at the right zone of curves type 2. This shallow minimum corresponds to a thixotropic gel, and represents the distance at which the particles arrange themselves in a soil deposited in a slow and quiet manner.

Let us therefore leave established that, in accordance with the laws of attraction and repulsion, the soil adopts by itself an equilibrium state, which sometimes is a single one (perfect coagulation in curves of type 1), and some other times is double (perfect coagulation or thixotropic coagulation, at either side of the energy barrier, in curves type 2). Soils cannot be displaced from this equilibrium position, except for the action of an external force.

This action can be moisture deficiency (insufficient liquid contents for development of a complete double layer). But we will now deal with the case when the external force is a mechanical one.

This force, if a compression, plays the role of an attractive force, and its potential is negative for decreasing distances. If we establish that the force is independent of the distance (a very common case), the curve representative of its potential is a straight line.

Now, adding this new potential to the two potentials mentioned above, we obtain a new curve for the total potential. This potential curve has a new minimum, which corresponds to a smaller distance between particles, as it could be reasonably expected. The construction of the new curve, as well as

the determination of the new minimum, are shown on Fig. 1. It can be seen that, for a certain value, the minimum becomes an inflexion point, with horizontal tangent (P_4). This corresponds to the crossing of a new "energy barrier." It could also be seen that the distance corresponding to this energy barrier varies with the form of the external force potential curve, which can be other than a straight line.

Fig. 1 also shows an "external pressure-distance" curve, similar to a consolidation curve, calculated from the potential-distance curve.

Let us now consider the shear strength problem, and examine positions of shear strength in Fig. 1.

We could assume that shear strength is proportional to the force required to produce a certain deformation, but this would be ridigity, not strength. Certain considerations, that the writer will explain in detail a little further, lead him to relate shear strength with the quantity of energy (ΔP in Fig. 1) required to bring the soil from its equilibrium position to the shallow minimum, representative of thixotropical coagulation. This energy increases with decreasing distance, i.e., with increasing external force. The curve "external pressure-ΔP" is also shown on Fig. 1.

In this light, the assertion posed in Mr. Lambe's paper that "the greater the attractive force, the greater the shear strength; the greater the repulsive forces, the lower the shear strength" (pg. 16), does not make much sense. As a matter of fact, the greater the external force, the greater the repulsive forces, since an equilibrium must be reached, but the shear strength will be greater too, and not lower.

Probably, Mr. Lambe's assertion must be interpreted in another sense: if we alter the electrical forces so that the repulsive force diminishes, the shear strength increases for a given external pressure. In Fig. 2, we have represented the potential-distance curves of a certain colloid, for various electrolyte concentrations in pore water. Curves 1, 2, and 3 correspond to increasing concentrations, and correspondingly decreasing repulsions. For higher concentrations of the electrolyte, attraction predominates over repulsion for every position of the particles, and the colloid coagulates with its particles in close contact (curve 4).

It can be seen in this figure that, for the same external force, the higher the electrolyte concentration—i.e., the lower the repulsion—the higher ΔP,—and let us suppose that the greater the shear strength, as Mr. Lambe advanced in his paper.

Let us bear in mind that the pre-consolidation process has no influence whatsoever upon all which has been stated hereto. After the action of a force disappears, the particles should immediately recover their original position in a perfectly elastic manner.

Pre-consolidation can only be considered in a statistical manner. To each external pressure, there will correspond a determined number of particles which, due to their special positions, will pass the energy barrier in an irreversible manner. The higher the external pressure, the higher their number will be. The irreversible orientation changes can give a statistical account of preconsolidation phenomena.

Now, let us consider the process of shear itself. We are going to follow the general lines stated by TAN TJONG KIE (1957), although this author gives much less importance than the writer to the physicochemical part of this phenomenon.

Clay, for the consistencies for which application of these theories can be

imagined, does not break as a brittle solid, but flows like a viscoelastic material. Some recent experiments are sufficiently conclusive to demonstrate that there exists an elastic limit, or first deformation threshold, before which the deformation is completely recoverable, and clay behaves as a perfectly elastic solid. The elasticity coefficient is very much higher at this stage than the elasticity coefficients or deformation moduluses ordinarily encountered in consolidation tests and unconfined or triaxial tests. For instance, DENISOV and RELTOV (1957) mention the case that clay masses, the deformation modulus of which does not exceed some 1,000 kg/sq. cm, show elasticity coefficients of up to 20,000-30,000 kg/sq. cm, when calculated starting from the propagation velocity of seismic waves, and the same authors were able to determine, by means of a vibratory plastometer, elasticity coefficients of several times 10 kg/cm^2, in clay pastes which under static load conditions purely behaved as viscous liquids. Likewise, GEUZE and TAN TJONG-KIE (1954) have been able to determine, by means of a special plastometer, the elasticity limit of some clays, under static load conditions.

This range of true elasticity could correspond to the stresses which are not able to alter the structure of clay. The particles move one with respect to the others, but they do not lose their bonds, i.e., they do not get farther apart than the distance corresponding to the shallow minimum of potential.

But, after the elasticity limit is surpassed, the bonds start sliding and breaking. Some of the bonds form again instantaneously in other points, due to the attractive forces between particles. In the event that the number of bonds created equals or exceeds the number of bonds broken, the material remains unaltered or practically unaltered, from a statistical point of view, in spite of the fact that it has undergone a great deformation, which is not recoverable.

If this is not the case, i.e., if the number of bonds which break is much greater than the number of bonds created, the structure disintegrates, the deformation is accelerated, resulting in what we call rupture.

It is thus explained why the breaking load depends upon the deformation speed, in those cases where the deformation is forced, and it justifies the writer's correlating shear strength with an energetic concept.

Nevertheless, it is the writer's opinion, as stated above, that we still are very far from finding a link between the physicochemical theory and the actual behavior of soils, most specially on the field of shear strength.

Let us study the following case as an example of the foregoing.

Several authors (HVORSLEV, 1938, SALAS and SERRATOSA, 1953, MITCHELL, 1956) have demonstrated by different means that a monoaxial compression orientates clay particles in a direction at right angles to the force direction. By imagining clay thus oriented, it is not too easy to see what shear strength the clay will show in the direction of the orientation, but, admitting its existence, it seems that it has to be lower than that perpendicular to the orientation, since in this direction it will be necessary to entirely disorganize the structure. We can thus easily demonstrate that the inclination of the shear planes must be entirely different (greater than 45^o with respect to the direction of the force applied) from those observed in practice, which proves how far theory is still from reality.

However, we can point out a case where theory can aid to interpret a fact observed. Let us admit that parallel orientation results in anisotropy as far as cohesion is concerned, this cohesion being smaller in the orientated direction.

At the initial moment of failure, the shear planes have a certain inclination as a result of the joint action of cohesion and friction (which with difficulty can be accounted for in physicochemical theory, in the case of parallel orientation). But, after failure is started, the particles orient themselves in the direction of flow, resulting in an orientation more complete even than that due to monoaxial compression. This results in a new pattern of anisotropy which determines the rotation of the shear plane, tending towards 45°, according as the shear test progresses. Now, the fact that the shear plane rotates in said direction has been observed by several authors, GIBSON (1953), for instance.

Let us point out that the anisotropy due to the orientation of the particles should also become apparent under other circumstances. Thus, if can be stated that the inclination angle of the failure plane in soils compacted on the wet side should be other than in soils compacted on the dry side, which might be useful as a suggestion for further research.

Finally, the writer would like to know whether the test conducted heating and cooling a sample of Boston blue clay, as described in page 11 of Mr. Lambe's paper, was carried out on a perfectly saturated sample. SALAS and SERRATOSA (1957) conducted a similar test on non-saturated clay samples, obtaining similar results, but they explained these results by the variation of capillary potential with temperature.

Through all these remarks, the writer does not at all intend to minimize the importance of Mr. Lambe's work, but he wishes to banish the feeling that the physicochemical theory has already solved the fundamental problems of soil behavior. At the present time, we should rather limit ourselves to point out that this is a brilliant way, and encourage researchers to follow it, in the hope that some day it will be possible to reach positive results in the solution of said problems.

On the other hand, this way is only useful to study "clay paste," however, a large part of soil behavior is due to the "Macromechanics" of clay aggregates and of clay-coarse particle aggregates (TERZAGHI, 1956), for the study of which theoretical ideas in the field of pure Mechanics are required. This is the reason why some researchers (SALAS and SERRATOSA, 1953; BOLT, 1956) have worked with extremely pure clay, in order to avoid the interference of these aggregation phenomena. These researchers did not pretend that the results of their tests be representative, in a general manner, of the behavior of the natural soils that the engineer must face, although these tests afforded basic facts for the interpretation of such behavior.

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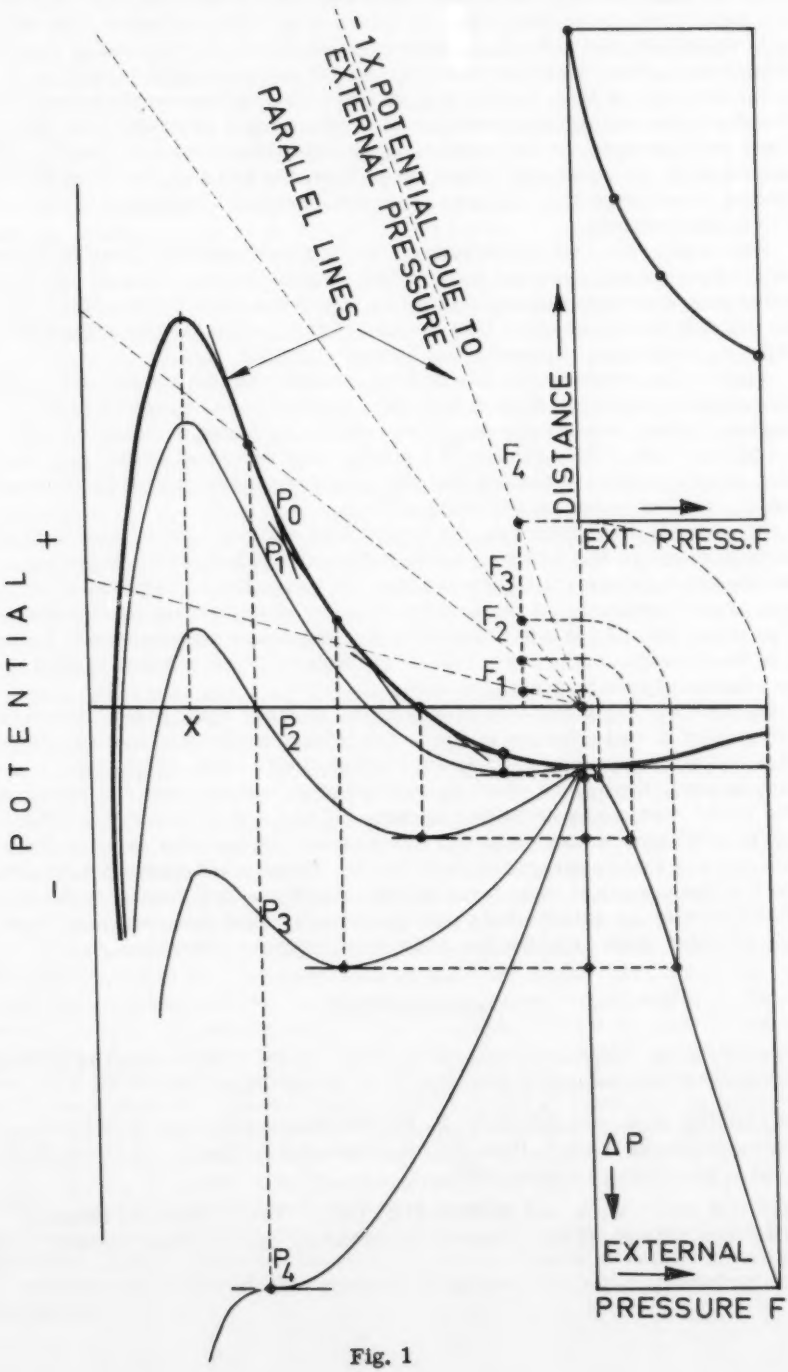
-1X POTENTIAL DUE TO EXTERNAL PRESSURE
PARALLEL LINES
DISTANCE
EXT. PRESS.F
F4
F3
F2
F1
P0
P1
P2
P3
P4
X
POTENTIAL
+
-
ΔP
EXTERNAL
PRESSURE F

Fig. 1

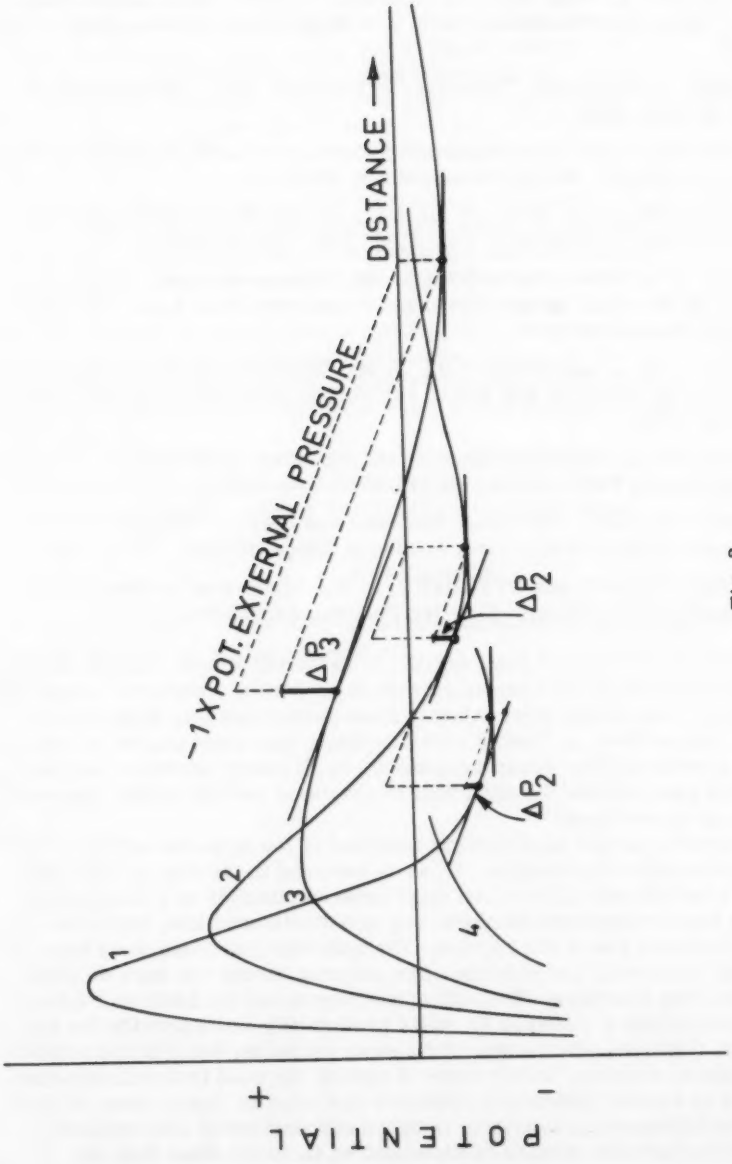
DISTANCE
-1 X POT. EXTERNAL PRESSURE
ΔP3
ΔP2
ΔP2
1
2
3
4
+
-
POTENTIAL

Fig. 2

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D. HUGH TROLLOPE,[1] A.M. ASCE.—In focussing attention on the reorientation of colloid particles associated with shear strain, Professor Lambe has contributed to one of the most valuable fundamental concepts of particle behaviour. Indeed there is limited visual evidence that some degree of reorientation is associated with shear deformation of all platey particles and that in the case of clay colloids the electrical interactions merely modify the resulting particle dispositions.

Consideration of the equilibrium conditions of the disperse system leads to some interesting conclusions. If, as is assumed in the theory under discussion, a net attractive force can exist between platelets in a disperse array, then this arrangement can have only one stable condition; that when all parallel particles are in full contact. The individual particles must come together and 'coalesce,' and it follows that external forces can have no effect on the resulting structure. The author has introduced the additional term—Contact Interaction (I - Figure 7), which presumably would provide the resistance to this complete merger of disperse particles, but it is not related to any physical quantity. In the writer's opinion the need to introduce some term such as Contact Interaction indicates that existing theory does not fully explain the interparticle behaviour in dense suspensions of clay colloids. The net force between colloids is dependent on factor(s) other than the

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electro static repulsions of the diffuse double layers and the attractions attributed to the long range Van der Waals forces. It appears that the laws governing contact interaction are likely to be of real importance in arriving at a fuller understanding of the true effective stress with respect to shear strength phenomena.

Following a study of the behaviour of saturated clay soils under isotropic three-dimensional consolidation, the writer's colleague Dr. R. H. G. Parry(1) has recently suggested that the net interparticle electric forces are not necessarily a unique function of interparticle spacing. An illustration of this is given in the case of the characteristic consolidation (p/e) curve where differing values of the consolidating pressure are required to maintain a selected value of the void ratio depending on the degree of overconsolidation. Whereas even under isotropic stress conditions this can be visualised as being associated with progressive reorientation of unsymmetrically arranged particles, this in itself does not appear to be sufficient to account for the experimental observations. Is it possible that contact interaction is only called into play when particles tend to approach each other more closely? A conceivable hypothesis to account for this would be that the water molecules and associated ions in the diffuse double layers, like the colloid particles themselves, are subject to reorientation under the consolidating influence of external forces and that this process is, at least partly, irreversible.

It is perhaps inappropriate at the present time to ask the author to elaborate on Contact Interaction, but it does appear to be of considerable importance and his views on this would be most welcome.

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PREDICTING SEEPAGE UNDER DAMS ON MULTI-LAYERED FOUNDATIONS[a]

Discussion by Harry R. Cedergren

HARRY R. CEDERGREN,[1] A.M. ASCE.—The authors are to be commended for developing mathematical solutions for estimating seepage through multi-layered foundations of impervious dams. Rigorous mathematical solutions for Laplacian equations, even for the simplest cases, are extremely involved, and practically impossible for cross-sections of any complexity. By introducing simplifying assumptions, approximate solutions may be obtained. In this manner the authors have obtained their equations. Undoubtedly there will be numerous practical applications for their formulas. They permit approximations of seepage without the need for skill in the construction of graphical flow-nets. Of course, graphical flow-nets that have been constructed with proper regard for the basic properties of the flow-net do represent theoretically correct solutions to the fundamental equations of flow. The flow-net has the additional advantage of presenting a visual picture of the pattern of flow and head loss throughout a section. Where it can be used it is unsurpassed as a means of evaluating the influence of various kinds of drainage systems on the safety and economics of dams and levees.

The authors place considerable emphasis upon the difficulty of applying the flow-net to the analysis of seepage under dams on multi-layered foundations. They state that "Even in cases where there is only one pervious and one impervious layer, a flow-net is impractical if there is a very great difference in permeability between the layers because of the need for drawing either extremely large or minutely small 'squares.' " Actually, any flow-net contains an infinite number of flow lines and an infinite number of equipotential lines, but the person constructing the net has the option of selecting only those lines needed for establishing the shape of the flow pattern. When a flow-net is being constructed for a cross-section having layers of widely different permeability, elongated rectangles can be freely used, provided the rectangles in each layer have the proper length-width relation with respect to those in other layers. An engineer's scale can be used very handily in keeping the length-width relations in the correct proportions. Occasionally, intermediate lines may be drawn to make sure that the proper relations do exist.

To explore the possibility of obtaining flow-net solutions to multi-layered foundations the writer constructed the three flow-nets that have been reproduced at considerably reduced scale in Figs. 1, 2 and 3. A one blanket-one aquifer (two layered) case is shown in Fig. 1; a two blanket-two aquifer (four-layered case is given in Fig. 2; and a three blanket-three aquifer

a. Proc. Paper 1727, August, 1958, by Paul H. Shea and Harry R. Whitsett.
1. Sr. Materials and Research Engr., California Div. of Highways, Sacramento, Calif.

(six-layered) case is given in Fig. 3. Only a portion of each net is shown, to permit use of the largest possible scale. All three solutions represent cross-sections with aquifers 120 times as pervious as overlying blankets. These flow-nets were all fairly difficult; however, the three were constructed in a comparatively moderate amount of time.

Continuing to explore the possibilities of using the graphical flow-net for the solution of seepage through multi-layered foundations the writer constructed a group of flow-nets in which the permeability of the blanket (upper layer) with respect to the permeability of the aquifer (lower layer) were varied through fairly wide limits. In the series, the permeability of the lower layer with respect to the upper layer varied from an upper limit in the order of 1000 to a lower limit of 0.01 or over a range of about 100,000 times. Additional flow-nets were constructed for zero permeability of the lower layer, to verify the general accuracy of the lower end of the series.

Shape factors that were obtained from this group of flow-nets have been plotted in a series of graphs that are reproduced at reduced scale in Figs. 4, 5 and 6. Data obtained from a larger scale plotting of these charts have been summarized in Tables I, II and III and may be used for reconstruction of the curves at larger scale. These charts give shape factors (n_f/n_d ratios) for impervious dams on two-layered foundations consisting of an upper layer with a thickness, z_1, a lower layer with a thickness, z_2, a combined foundation thickness, z, and a base width of dam (or levee), b. The charts provide shape factors for variable ratios of foundation depth to base width (shown as z/b) and for permeability ratios (k_2/k_1) from 0.01 to 1000. The variable relation between the thickness of the upper and lower layers has been introduced by provision of separate charts for each of three different values of the ratio of z_1 to z_2. Fig. 4 provides solutions for $z_1 = 0.1\ z_2$, Fig. 5 for $z_1 = 0.25\ z_2$, and Fig. 6 for $z_1 = 0.5\ z_2$. Shape factors for cross-sections not covered directly by the curves may, within reasonable limits, be determined by interpolation or extrapolation. The shape factors given by the charts are to be multiplied by the head times the permeability of the upper layer to determine seepage quantities. These charts provide a quick means for estimating the seepage quantity through two-layered foundations of relatively impervious earth dams and levees, and illustrate a practical but seldom encountered use of the flow-net.

Most soil deposits, natural or man-made, are more or less stratified. When the seepage analysis takes into account stratification, an appropriate transformation may be made in the horizontal scale of the cross-section[1,2] and the "effective" permeability must then be used. When analyzing multi-layered foundations, the best average transformation factor must be applied to the entire cross-section.

As an example of the use of the charts in Figs. 4, 5 and 6, a calculation will be made of the seepage quantity through the two-layered foundation of an impervious dam with a base width of 275 feet, resting on a 5-foot "blanket" of silty sand with an average horizontal permeability (k_x) of 4×10^{-4} ft. per min. and an average vertical permeability (k_z) of 1×10^{-4} ft. per min.; an underlying 50-foot "aquifer" of sand and gravel with average horizontal and vertical permeabilities of 0.04 and 0.01 ft. per min., respectively; and a head of 50 feet acting upon the dam.

The horizontal permeabilities are 4 times the vertical; hence a transformed section may be obtained by multiplying the base width by the

transformation factor, $\sqrt{k_z/k_x} = \sqrt{1/4} = 0.5$. Thus, the transformed base width, b, is 275' x 0.5 = 137 feet. The total depth, z, of the foundation is 55 feet, thus, the ratio z/b used in entering the charts is 55/137 = 0.40.

Continuing with the example, the effective permeability of the upper soil layer, k_1, is equal to $\sqrt{k_{1x}k_{1z}} = 10^{-4}\sqrt{4 \times 1} = 2 \times 10^{-4}$ ft. per min.; the effective permeability of the lower layer is equal to $\sqrt{k_{2x}k_{2z}} = \sqrt{0.04 \times 0.01} = \sqrt{0.0004} = 0.02$ ft. per min.; and the ratio of permeabilities of the lower layer to the upper layer, $\frac{k_2}{k_1}$, is 0.02 to 2×10^{-4} or 100 to 1.

The ratio of the thicknesses of the two foundation layers determines the chart that should be used. In this example $z_1 = 5$ ft. and $z_2 = 0.1\ z_2$, and Figure 4 can be used directly, without need for interpolation. From the curves in Fig. 4, a shape factor will be determined for the conditions of this problem, which are summaried as follows:

$$z_1 = 0.1\ z_2$$

$$\frac{z}{b} = 0.4$$

$$\frac{k_2}{k_1} = 100$$

From Fig. 4, a shape factor of 12 is read. The seepage quantity, q, for a one-foot length of this dam is calculated from Darcy's Law expressed by the formula:

$q = k_1\ h\ n_f/n_d$, in which

k_1 is the effective permeability of layer 1,

h is the net head acting upon the dam, and

n_f/n_d is the shape factor for the section, as determined from the appropriate chart.

Thus, q = (2×10^{-4} ft/min) (50 ft) (12) = 0.12 cubic feet per min. For a 1000 foot length of this dam, the calculated seepage quantity through the foundation is 0.12 (1000) = 120 cu. ft./min. or 9000 gallons per minute.

Many soils engineers seem reluctant to apply the flow-net to situations that are a bit out of ordinary. Their aversion is understandable. Undoubtedly, it is largely due to the difficulties encountered with new types of cross-section not previously solved. First attempts to apply the flow-net to new situations can be very time consuming, even frustrating or disheartening. Once the general seepage pattern has been established for a given type of cross-section, additional solutions usually can be obtained with greatly diminished effort. To illustrate this point, an experience of the writer will be noted. Flow-nets for earth dams or levees with an indeterminate phreatic line have been considered by "experts" to be rather difficult. Dams or levees on foundations with a permeability different from that of the structure, are considerably more difficult to analyze than embankments alone. The writer's first attempt to construct a flow-net for this type of section required over 10 hours of concentrated effort. After completing a number of flow-nets of this

kind, individual nets were obtained in a small fraction of the original time—some in as little as 10 or 15 minutes!

The purpose of this discussion has been to show that the flow-net is not altogether impractical for the solution of comparatively complex cross-sections such as for multi-layered foundations. The mathematical solutions developed by the authors certainly extend our sphere of knowledge. The authors' work is a noteworthy achievement, that undoubtedly will open the way to further solutions to seepage problems.

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TABLE I

FLOW-NET SHAPE FACTORS FOR
TWO-LAYERED FOUNDATION OF
IMPERVIOUS DAM (SINGLE AQUIFER)

$z_1 = 0.1\ z_2$

k_2/k_1	z/b 0.1	0.2	0.4	0.6	0.8	1.0	1.2	1.4
0	0.009	0.017	0.035	0.050	0.066	0.081	0.096	0.110
0.01	0.011	0.022	0.039	0.055	0.070	0.085	0.100	0.115
0.1	0.021	0.040	0.070	0.094	0.116	0.138	0.16	0.18
0.2	0.030	0.06	0.10	0.14	0.17	0.20	0.23	0.26
0.5	0.052	0.103	0.17	0.24	0.30	0.35	0.40	0.45
1.0	0.09	0.17	0.30	0.40	0.50	0.60	0.68	0.75
2.0	0.17	0.32	0.51	0.68	0.82	0.95	1.07	1.18
5.0	0.40	0.70	1.07	1.39	1.62	1.85	2.03	2.15
10	0.80	1.35	1.95	2.40	2.80	3.10	3.30	3.44
20	1.45	2.25	3.25	4.00	4.50	5.00	5.25	5.40
50	3.30	4.80	6.70	7.90	8.70	9.50	9.80	10.0
100	6.30	8.90	11.6	13.2	14.5	15.4	15.7	16.0
200	11.2	15.6	19.0	21.0	22.2	23.2	23.7	24.0
500	22.5	29.4	34.4	36.4	37.6	38.6	39.1	39.5
1000	34.0	41.0	46.6	48.8	50.1	51.1	51.6	52.0

z = total thickness of two-layered foundation, with upper layer of permeability, k_1, and lower layer of permeability, k_2. z_1 = thickness of upper layer; z_2 = thickness of lower layer.
b = total width of impervious dam.

TABLE II

FLOW-NET SHAPE FACTORS FOR
TWO-LAYERED FOUNDATION OF
IMPERVIOUS DAM (SINGLE AQUIFER)

$z_1 = 0.25\ z_2$

k_2/k_1	z/b 0.1	0.2	0.4	0.6	0.8	1.0	1.2	1.4
0	0.020	0.040	0.074	0.108	0.144	0.180	0.212	0.24
0.01	0.021	0.042	0.078	0.114	0.150	0.186	0.218	0.25
0.1	0.026	0.052	0.098	0.142	0.186	0.230	0.270	0.31
0.2	0.033	0.064	0.120	0.175	0.222	0.275	0.325	0.37
0.5	0.054	0.103	0.185	0.26	0.33	0.40	0.46	0.52
1.0	0.09	0.17	0.30	0.40	0.50	0.60	0.68	0.75
2.0	0.16	0.28	0.48	0.64	0.77	0.90	1.02	1.14
5.0	0.33	0.60	0.90	1.15	1.35	1.55	1.75	1.90
10	0.62	1.00	1.50	1.82	2.10	2.35	2.60	2.85
20	1.15	1.75	2.40	2.75	3.20	3.50	3.80	4.10
50	2.50	3.60	4.50	5.10	5.50	5.95	6.30	6.60
100	4.40	6.00	7.10	7.65	8.20	8.60	9.00	9.20
200	7.10	9.20	10.8	11.5	12.0	12.4	12.7	13.0
500	12.5	15.6	18.0	18.6	19.1	19.5	19.9	20.2
1000	18.0	21.5	24.8	25.6	26.0	26.4	26.7	27.0

z = total thickness of two-layered foundation, with upper layer of permeability, k_1, and lower layer of permeability, k_2. z_1 = thickness of upper layer; z_2 = thickness of lower layer.
b = total width of impervious dam.

TABLE III

FLOW-NET SHAPE FACTORS FOR
TWO-LAYERED FOUNDATION OF
IMPERVIOUS DAM (SINGLE AQUIFER)

$z_1 = 0.5\ z_2$

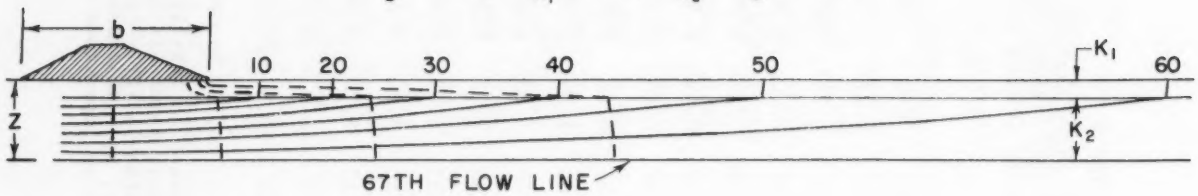
k_2/k_1	z/b 0.1	0.2	0.4	0.6	0.8	1.0	1.2	1.4
0	0.033	0.063	0.121	0.177	0.229	0.275	0.315	0.35
0.01	0.035	0.066	0.126	0.183	0.233	0.278	0.320	0.36
0.1	0.044	0.078	0.145	0.206	0.262	0.315	0.365	0.41
0.2	0.050	0.090	0.160	0.228	0.290	0.348	0.40	0.45
0.5	0.066	0.120	0.210	0.295	0.375	0.45	0.52	0.58
1.0	0.09	0.17	0.30	0.40	0.50	0.60	0.68	0.75
2.0	0.15	0.26	0.46	0.60	0.73	0.83	0.92	1.00
5.0	0.31	0.52	0.82	1.04	1.18	1.30	1.40	1.50
10	0.55	0.87	1.28	1.53	1.77	1.93	2.03	2.13
20	0.97	1.48	2.00	2.32	2.56	2.74	2.85	2.95
50	2.00	2.80	3.55	4.00	4.30	4.49	4.60	4.70
100	3.40	4.50	5.40	5.90	6.22	6.40	6.50	6.60
200	5.50	7.10	8.10	8.70	9.06	9.26	9.40	9.50
500	10.0	12.4	13.6	14.0	14.4	14.7	14.9	15.0
1000	16.0	18.5	19.8	20.5	20.9	21.2	21.4	21.5

z = total thickness of two-layered foundation, with upper layer of permeability, k_1, and lower layer of permeability, k_2. z_1 = thickness of upper layer; z_2 = thickness of lower layer.
b = total width of impervious dam.

FIG. 1

FLOW-NET FOR SEEPAGE UNDER DAM
ON MULTI-LAYERED FOUNDATION
(ONE BLANKET - ONE AQUIFER CASE)

$\frac{Z}{b} = 0.40$; $\frac{K_2}{K_1} = 120$; $\frac{n_f}{n_d} = \frac{67}{8} = 8.4$

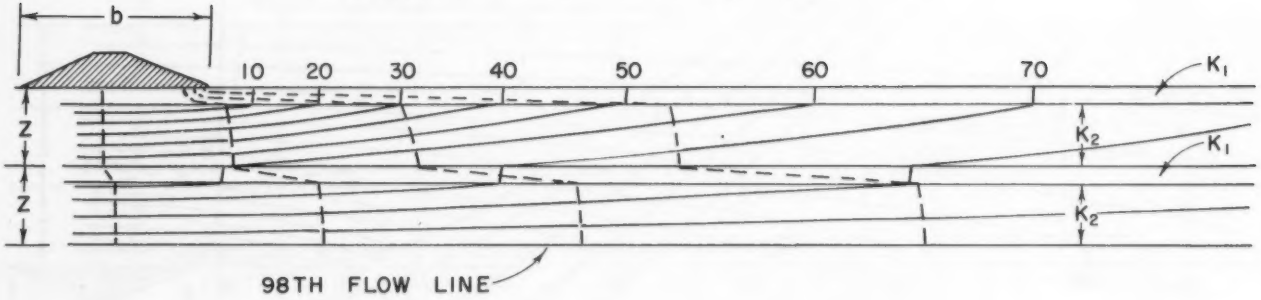


NOTE: SHAPE FACTORS AND RELATIVE SEEPAGE QUANTITIES FOR TWO BLANKET - TWO AQUIFER CASE ARE GIVEN IN FIG. 2, AND FOR THREE BLANKET - THREE AQUIFER CASE IN FIG. 3.

FIG. 2

FLOW-NET FOR SEEPAGE UNDER DAM
ON MULTI-LAYERED FOUNDATION
(TWO BLANKET – TWO AQUIFER CASE)

$\frac{Z}{b} = 0.40$; $K_2 = 120\,K_1$; $\frac{n_f}{n_d} = \frac{98}{7.9} = 12.4$

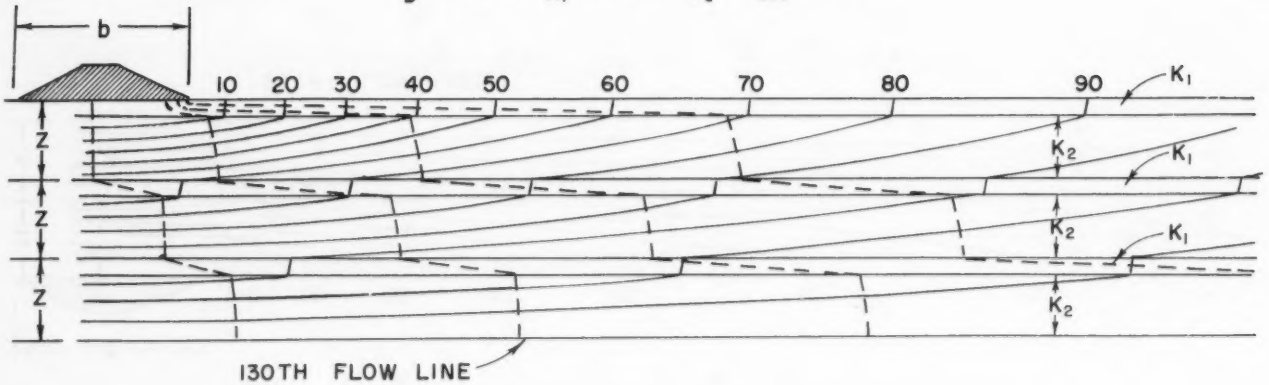


NOTE: SEEPAGE QUANTITY FOR THIS TWO BLANKET – TWO AQUIFER CASE IS 48% GREATER THAN FOR COMPARABLE ONE BLANKET – ONE AQUIFER CASE (SEEPAGE QUANTITY IS PROPORTIONAL TO SHAPE FACTOR, $\frac{n_f}{n_d}$).

FIG. 3

FLOW-NET FOR SEEPAGE UNDER DAM
ON MULTI-LAYERED FOUNDATION
(THREE BLANKET - THREE AQUIFER CASE)

$\frac{Z}{b} = 0.4$; $\frac{K_2}{K_1} = 120$; $\frac{n_f}{n_d} = \frac{130}{8.6} = 15$



NOTE: SEEPAGE QUANTITY FOR THIS THREE BLANKET - THREE AQUIFER CASE IS 90% GREATER THAN FOR COMPARABLE ONE BLANKET - ONE AQUIFER CASE (SEEPAGE QUANTITY IS PROPORTIONAL TO SHAPE FACTOR, $\frac{n_f}{n_d}$).

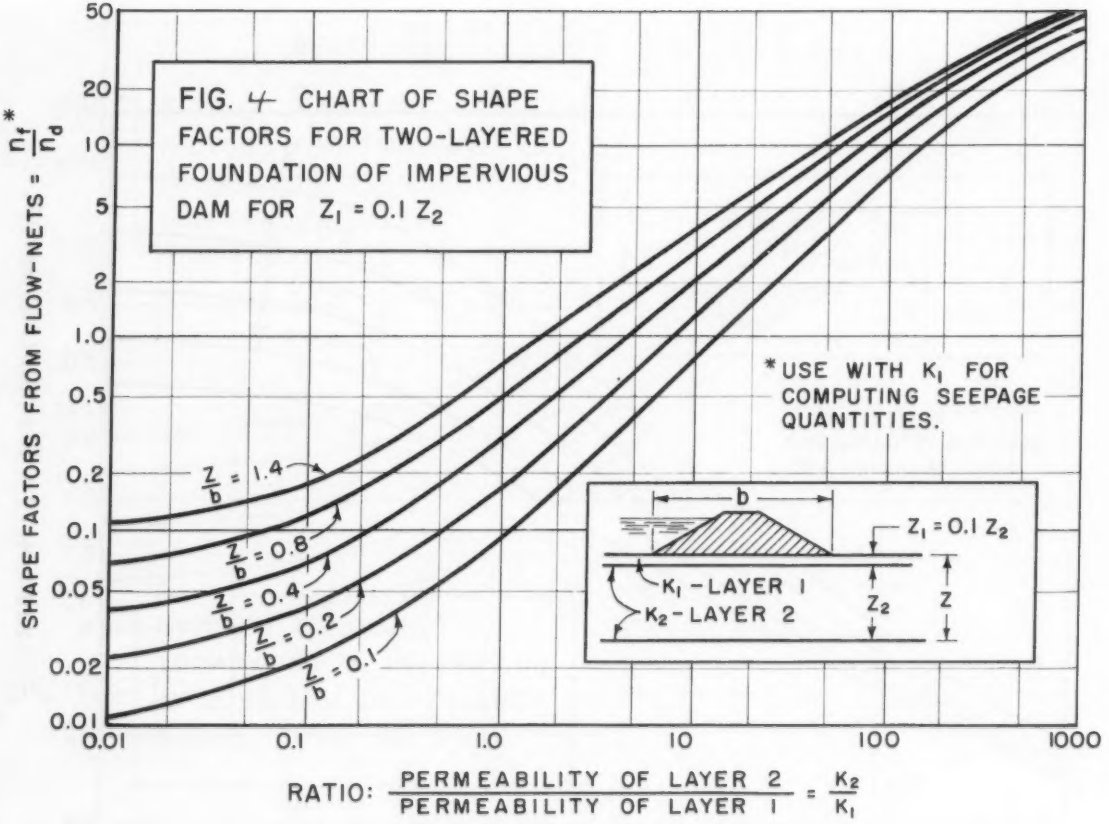


FIG. 4 CHART OF SHAPE FACTORS FOR TWO-LAYERED FOUNDATION OF IMPERVIOUS DAM FOR $Z_1 = 0.1\,Z_2$
SHAPE FACTORS FROM FLOW-NETS = $\frac{n_f^*}{n_d}$
50
20
10
5
2
1.0
0.5
0.2
0.1
0.05
0.02
0.01
0.01
0.1
1.0
10
100
1000
$\frac{Z}{b} = 1.4$
$\frac{Z}{b} = 0.8$
$\frac{Z}{b} = 0.4$
$\frac{Z}{b} = 0.2$
$\frac{Z}{b} = 0.1$
*USE WITH K_1 FOR COMPUTING SEEPAGE QUANTITIES.
b
$Z_1 = 0.1\,Z_2$
K_1 - LAYER 1
K_2 - LAYER 2
Z_2
Z
RATIO: $\frac{\text{PERMEABILITY OF LAYER 2}}{\text{PERMEABILITY OF LAYER 1}} = \frac{K_2}{K_1}$

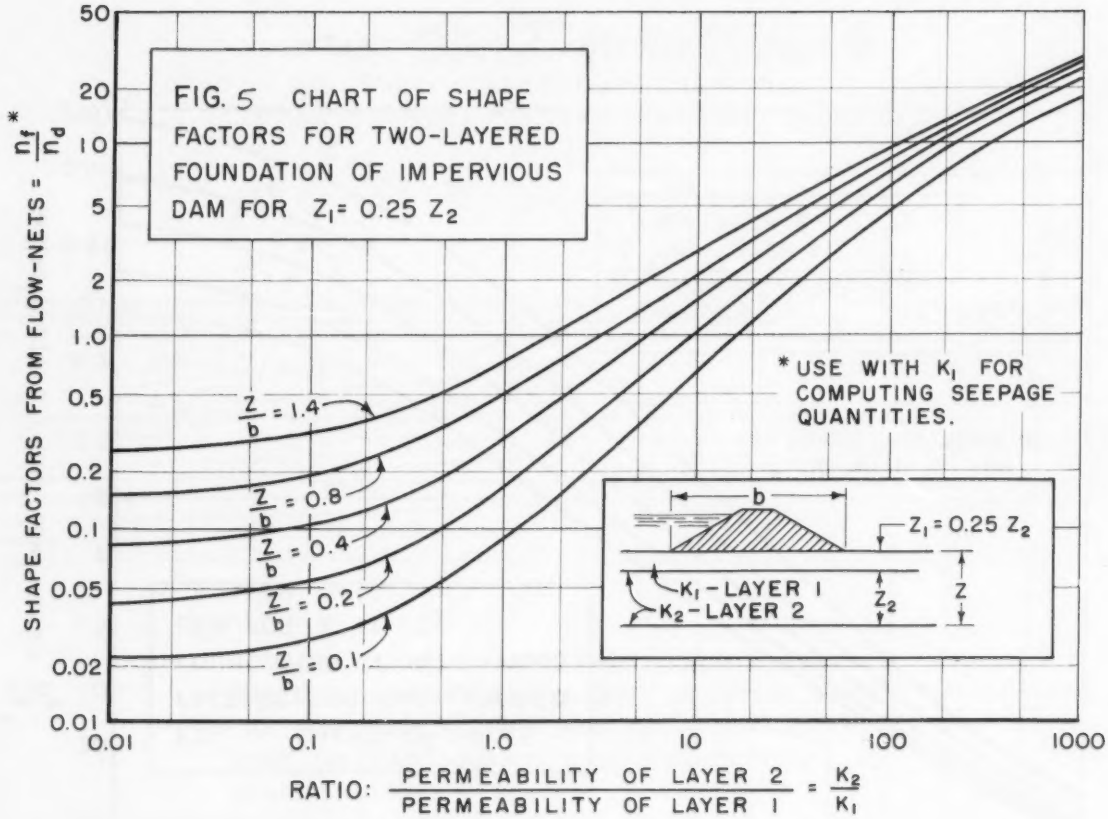


FIG. 5 CHART OF SHAPE FACTORS FOR TWO-LAYERED FOUNDATION OF IMPERVIOUS DAM FOR $Z_1 = 0.25\ Z_2$
SHAPE FACTORS FROM FLOW-NETS = $\frac{n_f}{n_d}$*
50
20
10
5
2
1.0
0.5
0.2
0.1
0.05
0.02
0.01
$\frac{Z}{b}$ = 1.4
$\frac{Z}{b}$ = 0.8
$\frac{Z}{b}$ = 0.4
$\frac{Z}{b}$ = 0.2
$\frac{Z}{b}$ = 0.1
* USE WITH K_1 FOR COMPUTING SEEPAGE QUANTITIES.
b
$Z_1 = 0.25\ Z_2$
K_1 - LAYER 1
K_2 - LAYER 2
Z_2
Z
0.01
0.1
1.0
10
100
1000
RATIO: $\frac{\text{PERMEABILITY OF LAYER 2}}{\text{PERMEABILITY OF LAYER 1}} = \frac{K_2}{K_1}$

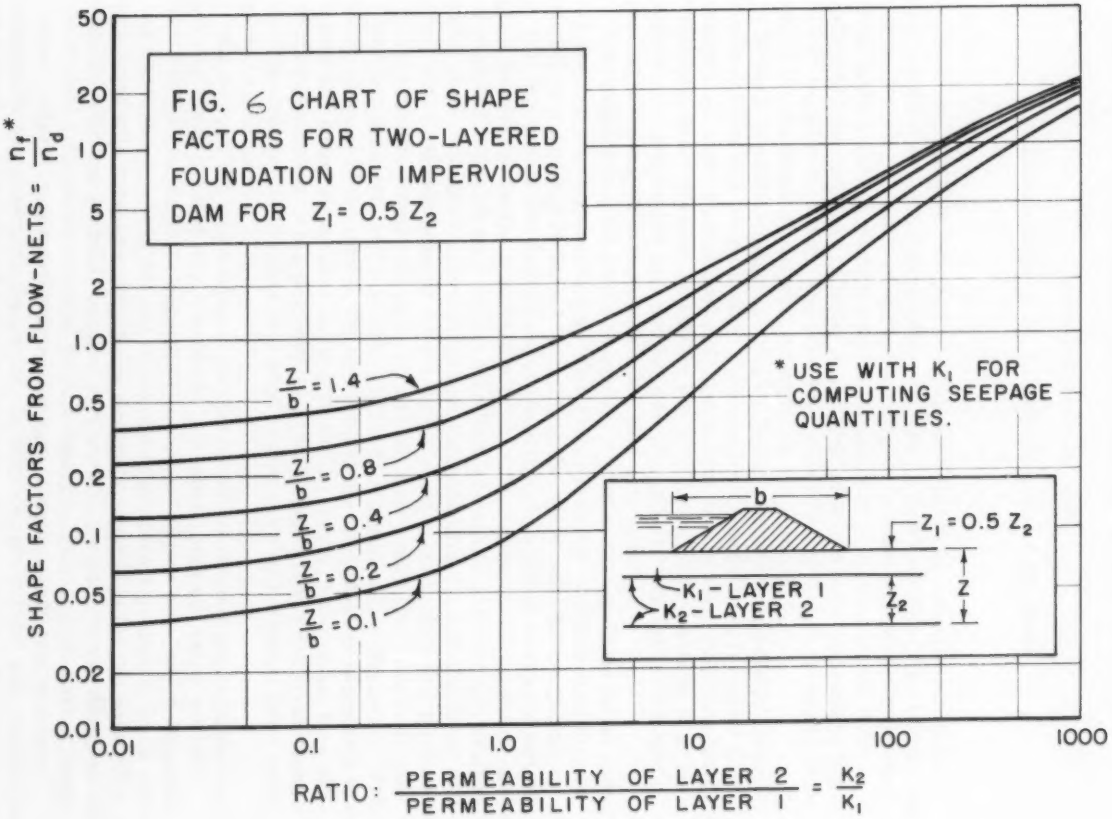


FIG. 6 CHART OF SHAPE FACTORS FOR TWO-LAYERED FOUNDATION OF IMPERVIOUS DAM FOR $Z_1 = 0.5\,Z_2$
SHAPE FACTORS FROM FLOW-NETS = $\frac{n_f^*}{n_d}$
50
20
10
5
2
1.0
0.5
0.2
0.1
0.05
0.02
0.01
$\frac{Z}{b} = 1.4$
$\frac{Z}{b} = 0.8$
$\frac{Z}{b} = 0.4$
$\frac{Z}{b} = 0.2$
$\frac{Z}{b} = 0.1$
* USE WITH K_1 FOR COMPUTING SEEPAGE QUANTITIES.
b
$Z_1 = 0.5\,Z_2$
K_1 - LAYER 1
K_2 - LAYER 2
Z_2
Z
0.01
0.1
1.0
10
100
1000
RATIO: $\frac{\text{PERMEABILITY OF LAYER 2}}{\text{PERMEABILITY OF LAYER 1}} = \frac{K_2}{K_1}$

DESIGN AND PERFORMANCE OF VERMILION DAM, CALIFORNIA[a]

Discussions by M. L. Dickinson and J. R. Kiely

M. L. DICKINSON,[1] M. ASCE.—The authors are to be commended for their clear and concise presentation to the profession of worthwhile information concerning solution of the many complex and unusual problems encountered in the planning, design, and construction of Vermilion Dam. The writer had the pleasure of being associated with this most interesting project throughout its design and construction but finds little to comment on or add to the thorough treatment presented by the authors.

By way of emphasizing the complex heterogeneous and variable nature of the foundation and the materials available for construction, it seems worth pointing out that construction of a dam of this size at this site would not have been considered feasible relatively few years ago. The advances in knowledge of soil mechanics engineering and its application to the interpretation of geology were major factors in making this dam possible.

The authors include only a brief description of the low level outlet extending through the dam foundation. The writer believes that this structure is worthy of additional comment concerning the somewhat unusual problems of soil mechanics and foundation engineering. The outlet works consist of: (1) a submerged intake structure with trashracks and an oil-pressure operated shutoff gate, (2) an 8-foot inside diameter modified horseshoe section reinforced concrete culvert, housing a free-standing 54-inch diameter steel pipe, which extends beneath the maximum section of the dam in a trench excavated into the foundation parallel and adjacent to the river channel, and (3) an outlet control valve and stilling basin structure. See Figure A. Prior to the closure of the intake, the culvert and the first stage construction of the intake and outlet structures were used for diversion of the river, thus permitting placement of the plug section of embankment over the river channel and construction of the dam as a single continuous embankment from abutment to abutment.

Because of the large depth of extremely heterogeneous glacial debris comprising the foundation above bedrock, the settlements and other possible movements that might be expected in a culvert of this size, which supports the maximum height of fill and is subject to reservoir pressure, could not be predicted with complete assurance. In order to provide for the possibility of considerable and probably unequal movements, the culvert was designed with watertight articulated joints at 40-foot centers. These joints permit longitudinal, lateral, and rotational movements of a magnitude that was considered to be conservatively ample. In addition, the culvert was constructed on a continuous downward grade from intake to outlet, with a 3-foot vertical

a. Proc. Paper 1728, August 1958, by K. Terzaghi and T. M. Leps.
1. Chf. Hydr. Engr., Bechtel Corp., Los Angeles, Calif.

OUTLET WORKS

FIGURE A

chamber above a straight-line gradient to allow for settlement. It is interesting to note that these provisions were more than ample since no important cracking has occurred and the maximum settlement of 0.27 foot has been quite nominal.

Wellpoints were set in the fill adjacent to the culvert at intervals throughout its length in and downstream from the core and were connected to gages inside the culvert. These provide a convenient means for observing the profile of the seepage pressure gradient through the embankment at its maximum section.

The remotely controlled 48-inch fixed dispersion cone type regulating valve at the outlet was selected, among other reasons, for its good energy dissipation characteristics, this being an important factor because of the highly erodable foundation.[1] The valve discharges into a concrete stilling basin, with an unlined discharge channel leading to the river. A deposit of poorly graded fine sand, having a variable depth up to several feet, was excavated from the bottom of the discharge channel just downstream from the stilling basin and was replaced with well graded sand and gravel to provide a filter for possible upward seepage emerging at this location and to protect against erosive action of the valve discharge. The channel waterway is riprapped with boulders for a distance of 150 feet downstream from the valve structure. These provisions have proved to be entirely effective, as essentially no erosion of the channel or movement of the riprap has occurred at this location.

J. R. KIELY,[2] M. ASCE.—This paper presents a frank and informative discussion of the many unique problems encountered in the design of a major earth dam construction upon a complex foundation of deep heterogeneous glacial materials with no possibility of providing a positive cutoff to bedrock. It should be helpful to design and construction engineers and geologists as an illustration of the type and magnitude of problems to be expected under similar circumstances and of satisfactory solutions devised by using sound judgment on the basis of substantial evidence without fully positive proof. The writer heartily concurs in the authors basic conclusions, in brief: (1) that the extent of the explorations were adequate and that no practicable amount of additional explorations would have been of significant value in more fully revealing the nature of the foundation and the borrow materials, (2) that under similar circumstances the designs and the construction requirements must be kept flexible so as to be readily adaptable to best suit conditions as they are actually found during construction, and (3) that Vermilion Dam and its appurtenant structures as constructed provides positive assurance of complete safety under all conditions of operation.

The authors have performed a special service to the profession by including a discussion of the changes made necessary by unexpected conditions revealed during the construction, which type of information is too often omitted from otherwise excellent papers. The authors description of these conditions and the measures adopted to best accomplish the intended purposes in an

1. This valve installation is discussed in Proc. Paper 1725, "Outlet Structures for Fixed-Dispersion Cone Valves," by Maurice L. Dickinson, Stanley M. Barnes, and Robert S. Milmoe, Jr., published in the August 1958 issue of the Journal of the Hydraulics Division.
2. Sr. Vice-Pres., Bechtel Corp., San Francisco, Calif.

economical manner reveals the close cooperation between engineering and construction which permitted desirable changes to be made promptly. The writer believes that the organizational arrangements under which the work was performed with the full responsibility for all engineering and construction under a single organization contributed greatly to the expeditious handling of the many unforeseeable conditions encountered.

PROCEDURE FOR RAPID CONSOLIDATION TEST[a]

Discussion by Icarahy da Silveira

ICARAHY DA SILVEIRA,[1] A.M. ASCE.—This paper presents very interesting and theorectically correct solution for the problem of obtaining consolidation results by a rapid procedure. The writer would like to add some comments and suggest methods of checking the practical results.

The first question is that of identification of the points in the consolidation curve, because only the 0% are experimentally determined (parabolic curve). Two ways were used: (a) neglecting the secondary effect in actual consolidation curves, the ratio (t_x/T_x) between time and time factor would be constant, because

$$\frac{T_x}{t_x} = \frac{c_v}{H^2}$$

and (c_v) as (H) are assumed as constants in computations.

This procedure was used by the writer before[2] with good results. Statistical methods are available to examine discrepancies. (b) a theoretical consolidation curve can be obtained point by point through the time reference (t_{ref}) in actual curve and the formula

$$t_x = T_x \frac{t_{ref}}{T_{ref}}$$

The ratio (T/T_{ref}) is obtained in a table prepared previously for different (x) values from 5 to 95% consolidation. Such tables are prepared for T_{ref} values of 50%, 70% and 90%, for different initial hydrostatic pressure diagrams.

Thus a theoretical curve is obtained to coincide with the actual test curve for checking purposes.

The second question is on the use of the steepest slope method for fixing the final reading in each consolidation step. The procedure was carrying the test to 70% of consolidation or little more for each load increment.

The Taylor method is another way without the use of the steepest slope, but 90% consolidation is necessary and drawing the curves as soon as the readings are obtained. The formula for any point of the curve is:

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a. Proc. Paper 1729, August, 1958, by Hsuan-Loy Su.

1. Cons. Eng. Comissao de Solos e Fundações - Rio de Janeiro - Brasil.
2. I. da Silveira - Consolidation under Special Load Condiction - Proc. 4th Int. Conference Soil Mechanics and Foundation Engineering - 3a/37 Vol. 1 - London 1957.

$$a_x = a_o + (a_{90} - a_o)\frac{x}{90}$$

Adding each load increment just after the 90% point the writer obtained good consolidation curves (pressure vs. readings), assuming x = 100%.

This extrapolation is justified by avoiding the introduction of considerable amounts of secondary effect as when consolidation is carried to 150% or more (24 hours test) as referred by the author.

The point a'_{100} corresponding to the load increment (p') must be the same as a''_o corresponding to the following load increment (p''), successively. It is important as a total load was considered by its effect without computation of residuals.

This alternative method demands greater time than that suggested by Mr. Su, because 90% is obtained with 1.94 t_{70} (see Table -1 col. T/T_{70}) but it is successful yet when compared with usual procedures.

Table 1 was prepared to be used in case of linear variation of excess initial hydrostatic pressure.[3] For another initial pressure the convenient consolidation curve must be elected, and the T/T_{ref} determined.

3. D. W. Taylor - Fundamentals Soil Mechanics - Wiley 1955.

TABLE - 1

U%	T	T/T_{50}	T/T_{70}	T/T_{90}
0	0	0	0	0
5	0.0035	0.0179488	0.008642	0.0041274
10	0.008	0.0410257	0.019754	0.009434
15	0.017	0.0871795	0.041976	0.0200472
20	0.031	0.1589744	0.076544	0.0365567
25	0.049	0.25130	0.120741	0.0577831
30	0.072	0.36923	0.177778	0.08491
35	0.097	0.49744	0.239507	0.11439
40	0.126	0.64616	0.3111112	0.14859
45	0.159	0.81539	0.392593	0.18750
50	0.195	1.00000	0.481482	0.23232
55	0.238	1.22051	0.587655	0.28067
60	0.287	1.47180	0.708642	0.33845
65	0.342	1.75385	0.844445	0.40331
70	0.405	2.07593	1.000000	0.4776
75	0.475	2.43590	1.172840	0.56015
80	0.565	2.89744	1.400000	0.66628
85	0.684	3.50770	1.688889	0.80661
90	0.848	4.34872	1.93828	1.00000
95	1.127	5.77949	2.78281	1.32901
100	∞	∞	∞	∞

EFFECT OF GROUND ON DESTRUCTIVENESS OF LARGE EARTHQUAKES[a]

Discussions by A. A. Eremin and N. N. Ambraseys

A. A. EREMIN,[1] A.M. ASCE.—Mr. Duke in this paper has shown the effect of seismic force distruction of structures based on the soft ground and on the rock formation. From the Martin's investigation Mr. Duke concluded that the earthquake shocks are more readily felt through the rock formation but the structures based on the rock foundation are the least affected by the destructive seismic forces.

Mr. Duke in his analysis has considered wooden structures, masonry, and rigid frame buildings. However, the classification of structures according to their rigidity properties was not described conclusively, except for the wooden building located in Japan.

It is notable that the statistics indicate clearly that the rigid frame structures are more resistive to the earthquake shocks. Furthermore, from the statistics the conclusion may be drawn that the structures based on a rigid slab and soft foundation would be more resistive to the seismic forces than those based on spread footings.

The relative extent of the propagation of the seismic force from the fault line may be drawn from the statement made by Mr. Wood. According to Mr. Duke's citation, Mr. Wood stated,

"Damage has some relation to distance from the fault line. However, the effect of kind of ground was more pronounced."

It is notable that the relative constants for the seismic forces on the structures based on various ground formations recommended by Mr. Takahasi, shown in Mr. Duke's citation, are the same as those used in design of bridges by the California State Bridge Department.[2]

Mr. Duke deserves sincere appreciation for his valuable study and analysis of the statistics of the earthquake destructiveness.

N. N. AMBRASEYS,[3] J.M. ASCE.—The author has presented a very interesting resume of the effects of the ground on the destructiveness of strong earthquakes.

In so far as rigid foundation material is concerned, his views are quite acceptable. In dealing with soft overburdens, however, the author has not considered the findings of Soil Mechanics as applicable to the problem. In the following these will be briefly discussed.

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a. Proc. Paper 1730, August, 1958, by C. Martin Duke.

1. Associate Bridge Engr., Californis Bridge Dept., Div. of State Highways, Sacramento, Calif.
2. "Highway and Railroad Bridges," A. A. Eremin, 1956.
3. Dept. of Civ. Eng., Imperial College of Science and Technology, London S.W.7, England.

In considering the age of an overburden as a soil parameter, the writer believes that this, for cohesive soils, is rather equivalent to investigating the bearing of two types of soils on the destructiveness of strong earthquakes, namely of normally consolidated and overconsolidated strata.

Since for the majority of the circumstances met in practice both types of soils are found in-situ saturated, their strength during an earthquake will have an upper limit which will be equal to that of their undrained unconsolidated strength c_u with $\phi_u = 0$. However, since the mode of deformation of the foundation material during an earthquake is of a repetitive type, thixotropic effects are present and for normally consolidated materials the strength of the soil during seismic loading will rather be the one corresponding to the remoulded state, c_{ur}. For a material with sensitivity greater than the factor of safety of the structure with respect to shear failure of its foundation material based on c_u, the remoulding effect of a strong ground motion may lead to failure. For foundation materials of high sensitivity, the danger of sinking or unequal settling of the foundation is evident.(1)

What has been considered, however, as an upper limit for the strength of saturated cohesive soils during rapid shearing, namely c_u or c_{ur}, holds only for loadings applied in one direction only. In shear oscillation problems the material is strained rapidly not only in one direction but in a series of full cycles of rapid loadings with shearing forces reciptrocating throughout the soil mass. The undrained strength of a saturated undisturbed soil previously exposed to such a pattern of shear will be less than or equal to c_u.

Another type of loading which bears some resemblance to the seismic shear distortion of a soil mass is that of the repetitive loading.(2) So far tests of this mode of loading on normally consolidated soils indicate that the soil when first deformed by repeated stress applications loses a part of its strength, and if the stresses are continuously applied, the soil shows large deformations. However, the majority of tests have been carried out on compacted semisaturated soils, and very little is known of the behaviour of a saturated undisturbed cohesive soil to such a disturbance. From tests of this type on compacted materials it appears however that for a certain type of soils a deviator stress of only 40% of the undrained strength of the soil was capable of inducing failure after several load repetitions.(3) It is however, impossible to infer from this type of tests on compacted materials the behaviour that may characterise saturated foundation materials during strong earthquakes. Quite apart from the differences in the tested and the foundation materials as well as in the mode of simulating the seismic disturbance and that of repetitive loading, the fact that the results from repetitive loadings are hampered by omissions such as measurements of pore water pressures and of the exact wave form of the repetitional load, does not allow any inferences to be made. The strength parameters of the materials deduced from shear strength results from repetitive loading tests are not of great value since they refer to behaviour in terms of total stresses.

Tests under reciprocating shear force can be carried out in the laboratory by triaxial testing of soil specimens. During the tests the center of the Mohr's circle to develop is kept immobile. This can be easily performed. After the sample is consolidated anisotropically, its initial stress state being represented in a stress plane by a circle, the confining pressure is made to flunctuate by a certain predetermined amount by means of a reciprocating piston connected to the cell. During this operation, the axial load is sustained on the ram through a hanger. By making use of a ram twice in area compared

to the area of the sample, we secure the immobility of the center of the circle of anisotropic consolidation, while its radius flunctuates with the confining pressure. Such an experimental arrangement secures the flunctuation of the shearing force without appreciable change in the angle of the plain of maximum shear. Tests on soils under this mode of loading, in the writer's opinion, represent the closest simulation of the actual shear oscillation to be imposed on foundation materials during seismic activity.

The hypothesis presented by various workers on the transient strength of soils, namely that the strength of soils irrespective of their stress history, increases during seismic distortion of the soil mass, is, of course, unrealistic. Seismic shear distortions are neither of the one way transient type nor of the order of strain rate which was employed in the tests which are responsible for the establishment of this hypothesis. According to the transient strength hypothesis one must expect even a sensitive clay stratum to exhibit a strength of nearly $2.c_u$ during an earthquake. This may be accepted for soils tested to failure in o.001 sec but it would be absurd to expect such conditions to obtain during an earthquake.

Cohesionless saturated foundation materials such as sands also show "ϕ=0" as for clays tested allowing no volume change during shearing.(4)

Vibrations of a stratum may be defined as high frequency disturbances being incapable of producing quasi-resonance in the stratum at a low mode, and consequently imparting to the material very small shear deformations in general. Oscillations, may be defined as such disturbances which would produce large shear deformations due to quasi-resonance of the stratum at lower modes, the fundamental included. However, sometimes it is not easy to distinguish between the damage inflicted upon a foundation material due to vibration and oscillation, since an earthquake produces both types of disturbances, in horizontal as well as in vertical direction. Vertical disturbances are in most cases smaller than horizontal ones and they represent vibrations rather than oscillations.

During an earthquake the vibrations are not sufficiently effective in setting up excessive pore pressures since their duration is comparatively short and their pattern non-regular. Oscillations, however, imparting to the soil masses large shear deformations, in particular in a horizontal direction, set up substantial excess pore pressures, practically within a second or two. It appears therefore reasonable to analyse the seismic behaviour of soil formations with a view to shear deformations produced by oscillations.

There is, however, under particular circumstances for a saturated compacted or in-situ granular soil in a rather loose state, an additional type of seismic disturbance—apart from that due to shear—which may cause change in the pore pressures and consequently in the effective stresses. This type of disturbance may result from compressional seismic waves travelling from the bed-rock upwards into the overburden and consequently for loosely packed granular materials this may result in increasing their dry density. The tendency of the void ratio to change during vertical vibrations sets up excess pore pressures, if the material is saturated, which in turn alter the effective stresses in the foundation existing before vibration. When the pore pressures, set up in a vertically vibrated saturated sand with free drainage, are positive, the effective stresses will increase as these pressures disappear with time. However, the effective stresses to develop when the pore pressures have dissipated will be exactly the same to those existing before vibration, the only alteration in the properties of the sand being that in dry density or water content.

Under undrained conditions, however, when the granular material is subject to shearing forces while vibrated vertically, the effective stresses to develop will depend also on the pore pressures due to the application of the shear force. The combination of vertical compressive with horizontal shear vibrations or oscillations is often employed in the design of compacting equipment.

In many instances it is expected for dense sands, when vibrated vertically, to show a small increase in pore pressure as a result from further densification of the material. In dense sands the sign of the pore pressures to develop depends on the intensity of the vibration. A dense sand may, at high vibrational accelerations, develop negative pore pressures due to the tendency of the soil's particles to move with respect to each other. When, however, the sand is sheared at the same time, the net pore pressure will depend on the magnitude of the shear strain imparted to the sand as well as on the intensity of vibration, and according to recent results, for comparatively dense sands, the net pore pressure becomes strongly negative at failure.

Investigations on large samples in the U.S.S.R.(5) on the pore pressures which may develop during vertical vibrations of a saturated sandy soil are in good agreement with similar tests carried out in Japan. However, these results cannot be applied to problems where failure of the vibrated mass will develop, accompanied by shear displacements. For very loose sands the actual positive pore pressures to develop during shear failure due to the combined effects of vertical vibration and horizontal shear will be much larger than those found in the tests in the U.S.S.R. From recent investigations on the matter(6) it appears that:

i) Vertical vibrations in comparatively loose saturated granular fills or foundation materials will set up positive pore pressures. These pressures may increase during simultaneous shearing of the material.

ii) Vertical vibrations in granular soils, when saturated will set up positive pore pressures. These pressures will decrease during shearing.

iii) Vertical vibrations in very dense saturated sands will set up negative pore pressures. Further decrease in pore pressure will result during a simultaneous shear deformation of the material.

From (i) and (ii) it follows that for saturated sandy soils liquefaction of the overburden may endanger the stability of any structure standing on it. The extent to which such foundation materials are vulnerable to liquefaction is, of course, a matter for investigation for every type of soil, compaction effort and expected intensity of the total disturbance.

A point which the writer is not aware of having been discussed in the past is the post-seismic behaviour of a foundation material which has survived a strong earthquake. The seismic shocks, obviously, imparted to the foundation material a certain amount of straining and the material is left with strains which in turn have set up a pore pressure to dissipate with time. The strained mass--being saturated--were subjected to shearing forces under conditions where no change in volume was permitted. Consequently the pore pressure is altered. The sign and magnitude of these changes in pore pressure will depend upon the stress history of the material. In normally consolidated or slightly overconsolidated soils the shear stresses will be

accompanied by an increase in pore pressure, while in overconsolidated soils the shear stresses will lead to decreases in pore pressure. After the earthquake, with the passage of time, these pore pressures will dissipate and the material will change in volume. Where the pore pressures were positive the volume will decrease, while negative pore pressures will lead to increases in volume.

The shear strength of the material depends on the intergranular stresses, and as changes in pore pressures cause equal and opposite changes in the intergranular stresses, the shear strength measured under undrained conditions, where no volume change is allowed to take place, will not in general be equal to the shear strength in drained condition, where the pore pressures have dissipated.

Consequently, after the earthquake, negative pore pressures set up during the passage of the disturbance will dissipate leading to an increase in water content of the soil and a decrease of the pre-seismic strength of the material. This will also lead to a swelling of the foundation. The amount of water to be absorbed, among other factors, will depend on the local straining and the developed pore pressures in the material. The dissipation of positive pore pressures will lead to a decrease in water content of the soil, and consequent increase in strength of the pre-seismic strength of the material. This will lead to a settlement of the foundation material.

It is, therefore, of importance to investigate the post-seismic strength of the material, in particular that of overconsolidated overburdens. The experimental investigation of such a phenomenon should require:

i) Anisotropic consolidation of the material under a particular stress ratio.

ii) Transient loading of the consolidated sample—shear stress controlled repetitive loading—with a force of an intensity prescribed by an analysis of the problem.

(iii) Dissipation of the pore pressures set up by the transient loading, and determination of the water content at equilibrium.

iv) Shearing to failure of the sample under constant minor principal stress and drained conditions.

In particular, soils should be investigated at principal stresses corresponding to their total factor of safety. At the moment, very little is known of the effect of such type of loading on soils, but from the evidence we have on tests of similar types of loading[7] appears that the behaviour of soils to such types of loading can be examined in the laboratory.

Another ground parameter used by many investigators is the depth of the overburden, and in many instances this has been used for the determination of the predominant period of the site. However, as one may expect, the rigidity of the foundation material varies with depth either due to the increase in the consolidation pressure with depth or due to both the layered structure of the formation and the overburden pressure. Also, in extensively built up areas, the predominant period of the overburden will be influenced by the structural weight standing on it. All these factors must be taken into consideration if predominant periods are to be investigated.[8] Undamped acceleration spectra often disclose such features of the overburden and it is of importance synchronisation of buildings in such periods as those of the overburden to be avoided.

In many instances it has been proved inconsistant to consider the seismic qualities of the ground independently from the type of the building to stand on it. Also, for a particular type of structure a comparatively weak foundation material may in many cases be more suitable than a rigid one. The writer hopes that Tables of Lateral Force Coefficients would not find their way into text books since such coefficients can not be standarised, being a matter for particular investigation for every individual structure and foundation material.

Another ground parameter is damping. Damping for soils is a factor on which few if any data are available. In soils, although it is rather difficult for the predominant type of damping to be found, one may assume that owing to the friction and sliding between particles, both types of viscous and frictional damping may exist. Frictional damping is rather unimportant due to its small influence on comparatively large strain amplitudes. This leaves the so-called "viscous" damping, which may in most cases be assumed proportional to the velocity of relative displacement as well as to the gradient of the rate of change of the relative displacement. Both types of "viscous" damping can be incorporated and expressed in terms of an equivalent amount of critical damping. There are indications that this may vary from 7% to 20%[9] "critical." However, since these values correspond to elastic distortions, they consist the lower limit for the damping. Hence, in actual destructive ground motions much higher values of damping may be developed. Although factors such as water content seem to affect damping considerably,[10] recent site investigations indicate a rather less important bearing of damping on such factors.[11]

Finally, the writer should like to know whether the author adopts the idea that:

i) The increase of soil strength that was exhibited in the dynamic tests for practically every type of soil would be developed during earthquakes.

ii) The alternating nature of the seismic motion will not influence the strength of the soil.

iii) The shear strain-rate in a soil mass during an earthquake would be of the order of the strain-rate which was used in the transient tests.

Also the writer wishes to know whether the author posesses any data on damping characteristics as well as on S-wave velocities in the plastic region of deformation of foundation materials.

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PRESSURE GROUTING FINE FISSURES[a]

Discussions by Bruce E. Clark, Herman L. Moor and Robert E. White
Corrections by Thomas B. Kennedy

BRUCE E. CLARK,[1] AFF. M., ASCE.—The importance of grouting the finest fissures in a dam foundation is sometimes questioned. It is true that the finest cracks seldom cause much leakage, and the cost of grouting them may well be more than the value of the water lost. However, small leaks have a way of increasing, and it is possible that many costly re-grouting jobs could have been avoided if the original grouting had filled finer cracks. The correlation of field experiences in grouting with the results of laboratory testing is usually difficult. The writer hopes that the following discussion may help to explain the differences between the author's test results and some common field observations.

One may well conclude, after a study of the author's thought-provoking paper, that past efforts to grout fine fissures have been somewhat misdirected. Although the importance of using the finest possible cement to grout fine cracks is fairly obvious, it has usually (with a few exceptions) been ignored. Instead, high pressure and very thin grouts have been expected to do the job, but the paper's chart of Fig. 4 shows that there is little relation between pressure and the minimum W/C at which grout will enter a crack. Also, the tests summarized on pp 19-20 show that high W/C grouts make a poor filling in fine cracks, and it was found that pressure could not force out the excess water (p. 34).

Reducing the maximum particle size of grout can be done by elutriation (in air or water) or by screening. Air separation is costly and apparently more effective at reducing the average particle size than the maximum (grain size curves given on plate 5 of(1) show about the same top particle size for a special 30 micron average size cement as the top particle size of the type III from which it was made). The writer once verified the fact that the top portion of a fairly high W/C grout allowed to stand 5 or 10 minutes contains much less coarse particles than the original grout, but there does not seem to be any practical way to take advantage of this fact in grouting fine cracks. At present, a vibratory screen appears to be the best way to reduce the maximum particle size of grout, within the limitations of screen sizes. In 1953, at Folsom Dam, the writer tested the capacity of an 18-inch circular vibrating screen, fitted with a 200-mesh stainless steel bolting cloth (approximately 50% more opening area than standard bronze screening). The following capacities were obtained:

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a. Proc. Paper 1731, August 1958, by Thomas B. Kennedy.

1. Dist. Geologist, Nashville Dist., U. S. Army, Corps of Engrs., Nashville, Tenn.

C/F Sx	W/C Weight	Flow CFM	Sx/hr
4.0	2.65	2.4	31
2.0	1.32	1.9	45
1.0	0.66	1.4	56

Apparently a fairly modest size screen—say 24 x 36 inches—would have enough screening capacity for most grouting jobs. A 240 mesh stainless steel bolting cloth is also available. Probably grout could be screened through a 325 mesh screen, but the capacity would be less than with 200 mesh. The benefits of screening grout do not stop at the separation of oversize particles, incidentally. Improperly mixed grout will not flow through a fine screen, so its use guarantees thorough mixing. Also, cement held in storage develops small lumps which do not mix (an early stage of warehouse set) and the screen rejects such lumps. Finally, the screen keeps out foreign matter from the grout, such as pieces of sack and lumps of set cement which may flake off the sides of the mixer.

The use of high grouting pressure certainly cannot be discounted altogether. The graphs of the author's Figs. 15 & 16 strongly suggest that below a certain pumping pressure a given grout would not flow at all. This behavior conforms to the flow characteristics of any plastic material, and is reason enough for using a considerable pressure when grouting fine fissures. But there is another reason for using high grouting pressures where possible: this is to consolidate the rock. Under pressure, the cracks taking grout tend to widen, allowing a better flow of grout in them, and squeezing together the cracks too fine to take grout. The writer believes that this consolidation of the rock formation is often the reason that increasing the grout pressure has so much influence on the rate of grout injection. Of course, pressure high enough to consolidate rock might also cause uplift and damage, so caution is desirable, particularly where there are horizontal openings or weaknesses in the rock.

The use of thin grout is always likely to result in an unsatisfactory filling of cracks, and the writer has seen grouted seams exposed in the field that looked considerably worse than that pictures in Fig. 18. However, the author's tests do show that a given grout will penetrate a finer crack at a high W/C than it will at a low W/C. Field grouting experience almost always shows that many holes which will take a considerable quantity of high W/C grout will almost immediately refuse when the W/C is lowered beyond a certain point. The field practice in such cases is, or should be, to try to find the W/C at which the most grout can apparently be forced into the ground. This would seem to be preferable to choking the holes with thick grout which can hardly get away from the hole at all.

The consolidation grouting done for the intake tower at Blakely Mountain Dam was an interesting case which showed that the use of high W/C grout does not necessarily result in poorly grouted cracks. The rock is more or less weathered "cemented" shale, dark grey-black when fresh, tan when weathered, with minor layers of quartzose sandstone. The rock lays steeply dipping to vertical, being close to the faulted axis of an overturned anticlinal fold. Because of its proximity to the fold axis, the rock is highly jointed, and consolidation grouting was considered essential to make it a satisfactory foundation for the 136 foot high tower. Because of the extremely jointed or "broken" character of the rock, it was found to be necessary to use extremely low grouting pressure. The pressures adopted, after some experiment, were:

Zone I - 5 lb.; Zone II - 10 lb.; Zone III - 15 lb.; Zone IV - 20 lb. The zones were 20 feet each, so the total depth grouted was 80 feet. Even at such low grout pressures, surface breakouts of grout were fairly common, and were treated by pumping a minimum W/C grout into the hole until it came back to the surface. The normal practice was to start grouting each hole with a 4.0 (CF/Sx)W/C grout, and cautiously reduce the W/C as far as was possible without prematurely plugging the hole. The following summary of grout mixes used shows that very little grout was pumped into the foundation at W/C's of less than 2.0 CF/Sx.

W/C	Total Sacks	Sacks Cement to plug surface leaks	Sacks in foundation
4.0	1519.9		1519.9
3.0	630.8		630.8
2.5	392.7	4.0	388.7
2.0	318.3	94.1	224.2
1.5	61.0	28.5	32.5
1.0	485.2	420.3	64.9
0.6	174.4	169.6	4.8

Now a 2.0 W/C, expressed in cubic feet/sack corresponds to a W/C of 1.32 by weight, which is about the same as that used to obtain the grout film of the author's Fig. 18. So it would most certainly be reasonable to expect, from the above tabulation, that the intake tower foundation was not well grouted, and that much of the grout would prove to be soft and spongy, and poorly bonded to the upper sides of the cracks. Because, then, there was considerable doubt as to the quality of the grout job done on the tower foundation, three 36-inch diameter calyx holes were drilled in it, two going down to the tops of the underlying diversion tunnels, and one between the tunnels to the level of the tunnel inverts. When the calyx holes were completed, the writer carefully logged them, going over the walls with a phenolphthalein solution, so as to detect the smallest seams of grout. There were no open, ungrouted cracks evident, and all the grout found was hard, and all the grouted seams were tightly filled with grout. Grout seams ranged from .005 ft. to .08 ft. thick. These facts were verified by Mr. Edward Burwell, then chief geologist with the Office, Chief of Engineers. The tower was then built, and when the writer was at the project several years later, there had been no signs of movement of the structures. The only explanation that the writer can see for the fact that the high W/C grout pumped lost its excess water, as it obviously did, is that the excess water escaped by bleeding, thus probably lowering the W/C of the grout to less than half its original value. Obviously, bleeding could not lower the W/C of grout in thin horizontal seams in the laboratory, nor, unless a series of vertical cracks were also present, would bleeding lower the W/C of grout in thin horizontal seams in a rock formation. One of the poorer grout jobs the writer has been was the consolidation grouting done under a concrete dam monolity which prevented the escape of excess water in the grout. It is evident that the nature of the openings being grouted and their location, attitude and degree of interconnection have a most important bearing on the difficulty of grouting them.

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1. "Grouting of Foundation Sands & Gravels," Tech. Memo. 3-408, Waterways Experiment Station, Vicksburg, Miss., June 1955.

HERMAN L. MOOR,[1] A.M. ASCE.—The author is to be commended on this thorough and thought-provoking treatise on a subject that is highly controversial despite the great mass of practical experience that has been accumulated. Much of this experience is the result of trial and error under actual job conditions, and despite wide differences in opinion as to best procedures under a given set of circumstances, most of the stated conclusions are generally accepted throughout the industry. Total water content of flowing grout is of no particular significance in comparison with water content of grout that has been deposited in ultimate position and composition. Excess water is merely the vehicle used for transporting grout solids to desired locations, and some water loss is normally anticipated in pressure grouting small crevices. However, neither pressure nor time can be relied upon for removal of excess water except in the limited boundary areas where considerable pressure differentials occur in a comparatively short grout travel distance. Under such conditions the fissure becomes clogged or "bridged" by the solid particles or flocs, and local pressure build-up causes further filtration and consolidation of succeeding particles or flocs until total sealing is achieved. Hence the pressure gradient from point of grout release to interior face of filtration seal represents only the slight loss due to internal friction and fluid friction of the grout, and the far greater pressure drop to approximately zero takes place in the zone of filtration, which may extend for an extremely short distance. With the low initial pressures of 25, 50 and 100 psi at point of release, the residual pressure for filtration and consolidation was well below that normally available for actual grouting operations. For solidification and maximum strength in the grouted mass, a comparatively heavy grout with low water/cement ratio is usually chosen despite the fact that better penetration and sealing can frequently be obtained with more water in the mix. The author does not state whether his prime objective was sealing the fissures or binding the mating surfaces together. Mixing time is not given, and the low-velocity mixer described will not result in anything approaching ultimate particle wetting and dispersion regardless of length of mixing period. Both calcium lignosulfonate and Intrusion-Aid reduce agglomerative tendencies of cement and thus assist in it's dispersion, but neither is a satisfactory substitute for a competent mixer. Each of the millions of individual flocs in the wet mix has a definite minimum energy demand for total particle dispersal, and a definite time limit before cementing effect renders further mixing even more difficult. The violent action of a high velocity mixer is far more effective in breaking down and homogenizing these flocs than the gentle stirring of the low speed mixer described. Grout screening techniques and secondary results are not stated, and it is interesting to speculate on proportion of material removed by screening as well as on the degree of floc breakdown effected by further agitation of masses which barely passed, or were forced through, the screen openings. When grouting between smooth mating surfaces 0.01, 0.02 and 0.03 inch apart, comparable results could be expected regardless of whether fissure was horizontal, on

1. Resident Engr., Lago de Guija, Harza Engrg. Co. International, El Salvador.

edge, or vertical with grout flow either upward or downward in channels of dimensions used in test. Natural fissures would require considerably greater linear separation of mating surfaces for equivalent flow capacity due to effects of partial particle interlock. Roughening test surfaces as described would increase bridging tendencies of partially or incompletely mixed grouts. With totally dispersed grouts the bridging tendency would be somewhat less, although still significant. Intrusion-Aid is normally, but not always, an effective fluidifier and expansive agent as well as helpful in reducing bleeding and increasing penetrative qualities. However, no chemical used as an admixture will materially assist in passing 0.012 inch particles through 0.010 inch openings. The need for better consistency determination and control is recognized. The torque meter is too delicate and timeconsuming to be practical for field use, and the messy and inconvenient flow cone or some similar time/flow-volume device is commonly used. Automatic viscosity control has been successfully applied to other industrial mixing processes, and undoubtedly the combination of a suitable sensing device with a simple and reliable control system would result in grout having negligible viscosity variation. In fact, a properly designed system would respond to consistency errors too slight to be detected even in the laboratory and still be rugged enough to withstand the abuse that is to be expected on most grouting operations.

ROBERT E. WHITE,[1] M. ASCE.—The author has made a valuable contribution to the art and science of grouting by reporting on the extensive laboratory tests which appear to have been made with care and imagination. Particularly he seems to present data which tends to dispel the long held notion that high pressures can be used to drive off excess water in thin watery grout in all cases. The author has made it clear that this applies to "the pressures and with the techniques used" in his program. No doubt the dimensions of the fissure and the permeability of the faces together with the extent of the pressure and the time factor all have a bearing on the drying off of excess water.

In recent literature on grouting, mention has been made of a method of producing grout which does not fall into any of the seven different grout categories reported on in the laboratory study. This method is the use of the high-speed, high-shear mixers or so-called "colloidal mills" which operate at speeds 1500 r.p.m. and over.

By the use of this type of equipment, it is believed the adsorbed molecules of air on individual cement grains are removed with consequent increase in gel formation and improved fluidity or "pumpability" of the grout for a given water-cement ratio. This, of course, would make for a stronger, more impervious grout. It is held that by this method of mixing, clumps or agglomerations of the solids are minimized—this characteristic would make for a grout better able to penetrate fine fissures for there would be less tendency for the agglomerations to bridge across openings of fissures and block them to further entrance of grout.

In preliminary laboratory tests on "the properties of grout which control its ability to penetrate and fill openings" Clark[1] presented the following table of the "Effect of Admixtures on Grout Properties"—

1. Vice. Pres., Spencer, White & Prentis, Inc. Engrs. and Contractors, New York, N. Y.

"Admixture	Flow Resistance	Plasticity	Mobility Retention	Bleeding
Bentonite (1%)	Increase	Increase	Reduce	Reduce
Clay (5-15%)	Increase	Increase	Reduce	Reduce
Diatomite (2-10%)	Increase	Increase	Reduce	Reduce
Lime (5-30%)	Little effect	Increase	Slight increase	Reduce
Calcium Lignosulfonate (1/4%)	Reduce	Reduce	Increase	Increase (usually)
High-speed mixing	Reduce	Increase	Increase	Reduce"

Since the effect of bleeding is deleterious, (as stated by the author it reduced bond to the top slab) a grout produced by high-speed mixing with reduced bleeding properties would be a superior grout.

In his closure (p. 1164) Clark states (among other requirements) that "Mixing should be thorough and at as high a speed as possible to promote thixotropic breakdown." His tests used a Colcrete centrifugal pump type of mixer.

Mayer,(2) in writing about the development of present French common practice of clay-cement grouting of sand and gravel formations, relates—

"The first [laboratory] tests on premixed alluvial materials failed completely. Cement alone, even when finely ground, just did not penetrate the sand and gravel. Cement mixed with stabilized fine clay behaved somewhat better but still penetrated only quite coarse materials. It was not until—[later]—that the question was taken up again. The tests in the field showed that even if the cement did not penetrate the layers of fine sand, the results achieved in the open gravel and the coarse sand were sufficient to yield an all round permeability of the order of 10^{-5} cm/sec., sufficiently low so as to practically suppress all leakage.

"So it came that what had been considered as a failure in the laboratory happened to be a success in the field. Two improvements also helped in obtaining good results; the high velocity mixers now used by practically all good grouting contractors (underscoring supplied) which change the granular composition of the cement and increase the stability of the grouts. Also the finely ground cement. . . . . "

Other authors, Kravetz(3) and the Engineering News-Record(4) write of the value of high-speed mixing.

Aside from the issue of grouting fine fissures, Colcrete high-speed mixers have been used extensively abroad since World War II for injected concrete of many kinds—underwater, high-density for nuclear shielding, penstocks, tunnels, paving, etc.(5) so that much experience has been gained in the operation of such equipment and the results obtained. A small amount of this work has been done in this country; one example is described by the writer in a recent article on the installation of Drilled-in Caissons.(6)

In view of the foregoing references, the writer suggests that fruitful research might be conducted on the use of high-speed mixers in producing grout for filling fine fissures. A further application of this research exists in the field of grouting cable channels for post-tensioning wires in prestressed concrete construction where a similar problem of filling small voids exists.

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3. Kravetz, Glebe A., "Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting" Proceedings, ASCE Paper 1546, pp. 5, 18, 21 & 23 (See also discussion by Clark, Proceedings October 1958, pp. 1828-25, 26 & 27.

4. "Mixing Machine Speeds Folsom Dam Grouting" Engineering News-Record, January 20, 1955, p. 45.

5. Champion, S. and Davies, Truman L. "Grouted Concrete Construction" Reinforced Concrete Association, London, England, February, 1958. 32 pages.

6. White, Robert E. "Grouted Stone Fills Caissons," Construction Methods and Equipment, December 1958, pp. 73-76.

CORRECTIONS—Footnote 1 on the bottom of page 1731-1 should be changed to read

1. Task Committee on Cement Grouting, Chf., Concrete Div., U. S. Army Engr. Waterways Experiment Station, Jackson, Miss.

PROGRESS REPORT ON GLOSSARY OF TERMS AND DEFINITIONS IN SOIL MECHANICS[a]

Discussion by Edward S. Barber

EDWARD S. BARBER,[1] A.M., ASCE.—While the difficulty of arriving at a generally acceptable set of definitions is appreciated, the writer feels that the following changes are necessary.

55. Slope of stress-strain curve is m_v (66). There is no need for a_v which is only used as $a_v/(1+e)$, thus causing unnecessary complication.

56. Use m_v in place of $a_v(1+e)$.

58. After "yielded" add "(consolidated)." Same for 120.

60. After "compressed" add "(swelled)." Same for 122.

61. Use symbol "f;" "f" not needed in 357, 358 and 366. Define as slope of curve representing relation between shearing resistance and normal stress. ϕ is usually calculated as arc tan f and then used to calculate f as $\tan\phi$; it would be simpler to use f throughout.

165, 305 and 354. The upper limit of sand should be the No. 10 sieve as used for international standards, highways, geology, agriculture, most bituminous work, building sand and in all but one of the current soil identification systems.

296. To be consistent, "density" should be substituted for "void ratio" in the definition; or the name should be changed to "relative void ratio."

325. From 324, silt size should include material between 0.05 mm and 0.074 mm.

380. Use m_v in place of $a_v/(1+e)$. H should equal thickness or other characteristic dimension. T is used for many drainage boundary conditions. For one dimentional drainage, H should be the thickness. The half-thickness idea is unnecessary and extremely confusing to students.

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a. Proc. Paper 1826, October, 1958.
1. Cons. Engr., Arlington, Va.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1) 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2). 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c] 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

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MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

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JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

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DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW1), 1928(HW1), 1929(SA1). 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 (CO1), 1959(CO1).

c. Discussion of several papers, grouped by divisions.

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SM 1

FEBRUARY 1959 — 8 NO. SM 1
VOLUME 85 PART 2

Your attention is invited

NEWS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION OF ASCE

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

February, 1959

NEWS FROM THE WEST COAST

The Soil Mechanics and Foundations Division of the San Francisco Section, A.S.C.E. have recently held two meetings. On November 5th, Mr. Earl B. Hall, who is in charge of the Soil Mechanics Laboratory for the South Pacific Division, Corps of Engineers in Sausalito, California, presented a program on "Pore Pressure Measurements." Mr. Hall is one of the best informed men in the United States on this subject since his laboratory conducts experimental work in this field at the Corps of Engineers. He reviewed the history of the work, commented on various types of equipment used both in the laboratory and in the field and demonstrated some of the recent cells with which he is working. Much of his work is concerned with extremely fine measurements for laboratory research programs. Other types of devices which are more suitable for field installations were also discussed. On December 10th Mr. James G. Patrick, Project Engineer of the Becthel Corporation presented a most interesting talk on "Soil Problems in the Construction of Swift Dam." The Swift Dam is part of a hydro-electric project in the State of Washington just a few miles from Portland, Oregon. The dam has just been completed and is probably the highest rolled earth fill dam in the United States. One of the more pressing problems in the construction of this project is the ability to handle rolled earth fill in a climate where the rainfall is relatively continuous. Mr. Patrick's talk was well illustrated with colored slides. This talk was similar to one delivered by Mr. Patrick at the convention held in Portland, Oregon in June.

Another program of great interest to soils engineers is to be delivered by John A. Bloom at a meeting of the Structural Engineers Association of Northern California. The subject is "Offshore Facilities." This talk will include the design and construction problems involved with offshore oil production installations. The particular project involves the actual construction of a man-made island in the Santa Barbara channel.

For the benefit of the people in the California area the Institute of Transportation and Traffic Engineering of the University of California are holding their Eleventh Annual Street and Highway Conference on January 28, 1959 on

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the Berkeley campus. The session will include discussions involving soils and pavements on road construction. Information can be obtained through the University of California.

MARTIN S. KAPP WINS RAYMOND AWARD FOR BEST PAPER ON FOUNDATIONS

Martin S. Kapp has won the first annual Alfred A. Raymond Award of $1000 for his paper describing a new technique of soil consolidation, it was announced by Maxwell M. Upson, chairman of the board of Raymond International, Incorporated, sponsor of the award. Named after the firm's founder, the award was established "to encourage originality in research and development in the field of foundation engineering.

Mr. Kapp, the soils and foundation engineer for the Port of New York Authority, described how a large area of unstable soil was consolidated by lowering the water table temporarily with wellpoints. The method was used instead of a conventional surcharge fill.

Entitled "A New Technique for Stabilizing Compressible Soils," the paper points out that lowering ground water level eliminated soil buoyancy, with the result that an overlying sand blanket increased in weight and effectively compressed an underlying stratum of soft silt.

Mr. Kapp attended Swarthmore College, Harvard College and the Graduate School of Engineering at Harvard University. He is active with the Metropolitan Section of the American Society of Civil Engineers.

Two honorable mentions were also awarded. One went to Mr. Charles W. Granacher of Dravo Corporation for a paper entitled "Construction of Main River Pier, Mississippi River Bridge, New Orleans," and another was awarded to James F. McNulty and James S. O'Brien, co-authors of a paper entitled "Underpinning a Wind Tunnel at Langley Field, Virginia." The co-authors are with the National Advisory Committee of Aeronautics, Langley Field.

Awards were presented at a dinner held in the Waldorf-Astoria Hotel. Winners were selected from among twenty-nine entrants by a panel of judges consisting of Professor R. B. Peck, foundation engineering, University of Illinois; Dr. R. E. Fadum, Head, Department of Civil Engineering, North Carolina State College of Agriculture and Engineering; and E. A. Dockstader, consulting engineer, Boston.

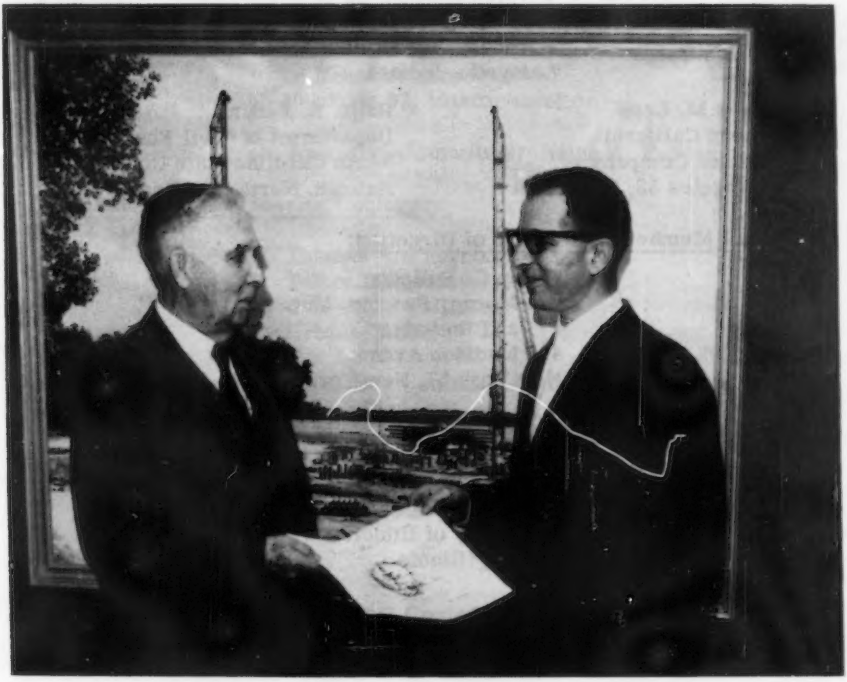
TEXAS SECTION SELECTS OFFICERS

The Executive Committee announces with pleasure the naming of the following men as officers of the Texas Section's Soil Mechanics and Foundations group:

Albert J. Bonar, Chairman, 3801 Cullen Street, Houston, Texas

Lymon C. Reese, Vice-Chairman, 173 Engineering Building, University of Texas, Austin, Texas

Raymond C. Mason, Secretary-Treasurer, 213 West Yorktown, Dallas, Texas



Maxwell M. Upson, chairman of the board of Raymond International Incorporated, presents certificate to Martin S. Kapp, winner of first annual Alfred A. Raymond Award for his paper on a new technique of soil consolidation. Mr. Kapp is a foundation engineer for the Port of New York Authority.

SOIL MECHANICS AND FOUNDATIONS DIVISION 1958-59 COMMITTEE ORGANIZATION

The following schedule of committee organizations of our Division is presented for your information. These committee men will appreciate your contacting them relative to any information of interest to the members of our Division.

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The Technological Institute
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School of Civil Engineering
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Raleigh, North Carolina

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Contact Member from Executive Committee: S. J. Johnson

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Omaha, Nebraska

Contact Member from Executive Committee: Tom Leps

Committee on Frost Action and Permafrost:

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99 Nehoiden Street
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Contact Member from Executive Committee: Ralph E. Fadum

Committee on Engineering Geology:

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Berkeley, California

Walter H. Price (Secretary)
U. S. Bureau of Reclamation
Denver Federal Center
Denver, Colorado

Contact Member from Executive Committee: Stanley J. Johnson

Committee on Road and Airfield Soil Problems:

Charles R. Foster
U. S. Waterways Experiment Station
Vicksburg, Mississippi

Contact Member from Executive Committee: Tom Leps

Committee on Glossary of Terms and Definitions:

Gerald A. Leonards
School of Civil Engineering
Purdue University
Lafayette, Indiana

Contact Member from Executive Committee: Ralph E. Fadum

Committee on Shear Strength Conference:

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U. S. Waterways Experiment Station
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VISITING LECTURER

Dr. I. Th. Rosenqvist, Professor, University of Oslo, and Head, Geochemical Division, Norwegian Geotechnical Institute, visited the United States and Canada in connection with the 1958 annual meeting in New York.

Dr. Rosenqvist presented a paper on "Clay-Water Systems," at the Symposium on Physico-Chemical Properties of Soils in New York, and visited and lectured at several universities during his stay, including Northwestern, Illinois, Purdue, Princeton, Harvard and Massachusetts Institute of Technology. Among his lectures were:

Rock Slides and Slide Control

Influence of Physico-Chemical Characteristics on the Mechanical Properties of Soils

Corrosion of Steel Foundation Piles in Soils of Marine Origin

Concrete Deterioration Due to Alum Shales in the Oslo Region

Sub-Polar Quick Clays

MONTHLY SCIENTIFIC PUBLICATION OF INTEREST TO SOIL MECHANICS DIVISION MEMBERS

It will be of interest to our Division members to know that the American Geophysical Union is combining the scientific and technological material in its bimonthly transactions with the quarterly JOURNAL OF GEOPHYSICAL RESEARCH as a monthly starting in 1959. This will contain much material bearing on Hydrology of interest to our members.

The subscription rate for the new Journal is $16.00 for the calendar year. Memberships, however, are invited. Membership is $10.00 per calendar year

and includes a subscription to the Journal. Any who are interested should write to the office of the American Geophysical Union, 1515 Massachusetts Avenue, N. W., Washington 5, D. C.

MISSISSIPPI HIGHWAY CONFERENCE

The fifth annual Mississippi Highway Conference will be held on February 26 and 27 at the University of Mississippi, Oxford, Mississippi. The conference will include: (1) a panel on Highway Design, moderated by Mr. W. J. Turnbull of the Waterways Experiment Station at Vicksburg, Mississippi (2) a demonstration of erosion control equipment (3) a panel on compaction equipment performance by representatives of the Waterways Experiment Station, contractors and engineers (4) a report on the AASHO Road Test at Ottawa, Illinois, and (5) a panel on Performance-type versus Quality-type Specifications by contractor, design engineer and contracts engineer representatives.

INVITATION TO PARTICIPATE IN INTERNATIONAL SOILS ACTIVITIES

The Soil Mechanics and Foundations Division of ASCE has announced a new opportunity made available to members of the Society through a consolidation of that division with the United States National Council on Soil Mechanics and Foundation Engineering.

The Division is setting up a roster which will amplify the list of members formerly enrolled in the U. S. National Council. Such roster will become, in fact, the U. S. national society, and a part of the International Society of Soil Mechanics and Foundation Engineering.

The primary purposes of the national societies are:

a) to solicit and process papers for international conferences;
b) furnish members for international research committees;
c) each year furnish the international secretary with a list of members, their occupations and addresses;
d) disseminate information on international affairs in soil mechanics and foundation engineering to its members;
e) cooperate with other national societies in collecting data in soil mechanics and foundation engineering.

In turn, the aim of the International Society is the promotion of international cooperation among scientists and engineers in the field of soil mechanics and its practical applications through:

a) periodically holding international conferences;
b) creating permanent research committees;
c) publishing a list of members every two years;
d) promoting the publication of abstracts.

All members currently listed with the U. S. National Council will be included in the new roster, automatically. Other members of the Society who wish to be listed on this roster, as well as on the roster of the Soil Mechanics and Foundations Division, may do so by sending their name, position, firm or organization and mailing address to the address following:

Mr. John Lowe, III
Secretary for International Activities
Soil Mechanics and Foundations Division, ASCE
62 West 47th Street (Room 400)
New York 36, N. Y.

Among the advantages of being listed on this roster are: the receipt of information on international conferences direct from the organization sponsoring the conferences, and the privilege of purchasing at discount certain publications of the International Society, and other national societies of this organization.

The Soil Mechanics and Foundations Division arranged this changeover to simplify administrative matters, and to provide broader representation of the engineers in the United States in activities of the International Society.

SOILS CONFERENCE IN COLORADO

On April 23, 1959, an all day Soil Conference on the subject Theoretical and Practical Treatment of Expansive Soils will be held on the campus of the Colorado School of Mines at Golden, Colorado, under the co-sponsorship of the Soil Mechanics and Foundations Division of the Colorado Section of ASCE and the Civil Engineering Department of the Colorado School of Mines. Speakers at this conference will include T. William Lambe, Head, Soil Engineering, Massachusetts Institute of Technology; Chester McDowell, Supervising Soils Engineer, Texas Highway Department; Raymond F. Dawson, Professor of Civil Engineering, University of Texas; R. E. Means, Professor of Architecture and Civil Engineering, Oklahoma State University; and W. G. Holtz, Chief, Earth Laboratory Branch, U. S. Bureau of Reclamation.

APRIL NEWSLETTER

Deadline date for arrival at this office of contributions for the April Newsletter: February 20, please.

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